Contract Report 608

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

by
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Office of Surface Water Resources: Systems, Information, and GIS

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Illinois State Water Survey
Hydrology Division
Champaign, Illinois

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CONTENTS

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INTRODUCTION

The primary goal of the Managed Flood Storage Option Project is to evaluate the benefits of converting at-risk levees and drainage districts to flood storage areas along the Peoria-Grafton section of the Illinois River. The implementation of this goal will involve an analysis of historical floods to determine flood magnitudes and frequencies at which overtopping of levees in this section of the Illinois River occurs. It will also require an evaluation of the impact and interaction of flow from the tributaries and backwater flow from the Mississippi River. The different components of the project can be grouped into two categories: flood frequency analysis and unsteady flow simulations.

The flood frequency analysis involves the development of discharge-frequency relations for Illinois River gaging station records at Marseilles, Kingston Mines, and Meredosia, and the gaging stations on the five major tributaries on the Illinois River, using both log-Pearson Type HI and mixed distributions. Also, stage-frequency relations have been developed for Illinois River gages at Peoria Lock and Dam (L&D), Kingston Mines, Havana, Beardstown, La Grange L&D, Meredosia, Valley City, Pearl, Florence, Hardin, and Grafton. The results of the flood and stage frequency analyses are used to develop 10-, 25-, 50-, and 100-year discharges and stages that are required in the unsteady flow model simulations. Singh (1996) reported the flood frequency analysis component of the project. The first phase of the unsteady flow modeling component is presented here.

Modeling of unsteady flow is based on the application of the UNET (HEC, 1993) hydrodynamic model originally developed by the U.S. Army Corps of Engineers (USCOE). The purpose of the unsteady flow modeling is to compute water surface profiles for the 10-, 25-, 50-, and 100-year floods using the hydrographs obtained from the stage-discharge frequency analyses (Singh, 1996) as boundary conditions. The water surface profile computations and model verification constitute the first phase of the unsteady flow modeling component and are reported herein. A third project report will include subsequent analyses of several levees in the La Grange Pool and a few in the Alton Pool for flood storage based on the current flood protection levels, maintenance costs and problems, and net agricultural benefits and economic viability. Each of the selected levees will be represented in the UNET model with an opening at the crown (500- to 1000-feet wide and 4- to 6-feet deep) so that floodwaters can flow into the levee district for temporary storage when the river stage exceeds the stage corresponding to about a 20-year flood. Water surface profiles will be generated for various combinations of candidate levees and cut dimensions. The simulation results will be used to develop guidelines for width and depth of cuts and to select those at-risk levees most suitable for conversion to managed flood storage options.

Since most of the levee districts along the Illinois River are below Peoria, the lower Illinois River from Peoria Lock and Dam (L&D) to Grafton was selected as the study reach Because of the pools created by the La Grange L&D in the Illinois River and Lock and Dam 26 in the Mississippi River at Alton, the study reach was divided into two segments: the upper reach from Peoria L&D to La Grange L&D (River Mile 157.85 to 80.1), and the lower reach from La Grange L&D to the Illinois River mouth at Grafton (River Mile 80.2 to 0). The UNET model was therefore set up as a system of two separate reaches (La Grange and Alton Pools) and as a single reach from Peoria to Grafton.

In order to apply the UNET model to predict floods in the lower Illinois River, the model parameters have to be calibrated to several historical flood events. In the analysis reported herein, the UNET model was calibrated by matching computed flood hydrographs at the location of the gages on the Illinois River with the observed flood records for 1979 and 1985. The accuracy of the calibrated parameters was then verified by simulating the 1973, 1974, 1982, and 1983 floods. As previously discussed, the next steps in the unsteady flow modeling involve the prediction of the 100-year and other frequency flood elevation profiles in the La Grange and Alton Pools, the reduction in the frequency and magnitude of flooding that would result from the impacts of the flood storage options, and the simulation of several scenarios involving combinations of at-risk levees and tributary inflow timings in order to evaluate the impact of the storage options on peak stages.

Hydraulics and Hydrology of the Lower Illinois River

The section of the Illinois River that is under consideration starts from River Mile (R.M.) 157.7 at Peoria to the confluence with the Mississippi River at Grafton. It is designated as the lower Illinois River. It is, for the most part, a winding waterway with wide, flat floodplains and steep bluffs on one or both sides. The channel varies from about 450 feet to 1100 feet in the La Grange Pool, and the valley width varies from about 2 miles near Pekin to about 5 miles near Beardstown. Downstream of Beardstown, the channel and valley are wider and there are more lateral lakes and ponds. The USCOE maintains a 300-foot-wide navigational channel.

Five locks and dams are operated on the Illinois River to maintain a 9-foot depth of channel for navigational purposes. Two of these locks and dams (Peoria and La Grange) fall within our study area (Figure 1). The only other lock and dam (L&D) that affects the lower Illinois River is L&D 26 on the Mississippi River, and 15 miles downstream from the mouth of the Illinois River. Peoria L&D and La Grange L&D were built between 1936 and 1939. The two structures have similar configurations and were designed with wicket gates that can be adjusted to maintain the 9-foot navigational depth. The wicket gates are lowered during high flows to create an open river condition. Both stuctures were modified between 1986 and 1990 by replacing some of the wicket gates with taintor gates. The wicket gates are now raised to full height during low flows and the water depth is controlled with the taintor gate. Figure 2 depicts the taintor gate structure and the wicket dam at Peoria. Figure 3 depicts the La Grange L&D.

The Peoria lock is located on the east bank of the Illinois River and is 100 feet wide and 600 feet long. Next to the lock is a taintor gate with 76-foot opening and a dam 436 feet long comprising 116 wickets that are 3.75 feet wide and 16.42 feet high. A concrete-regulating dam 34 feet long and having an abutment with a single inlet and six

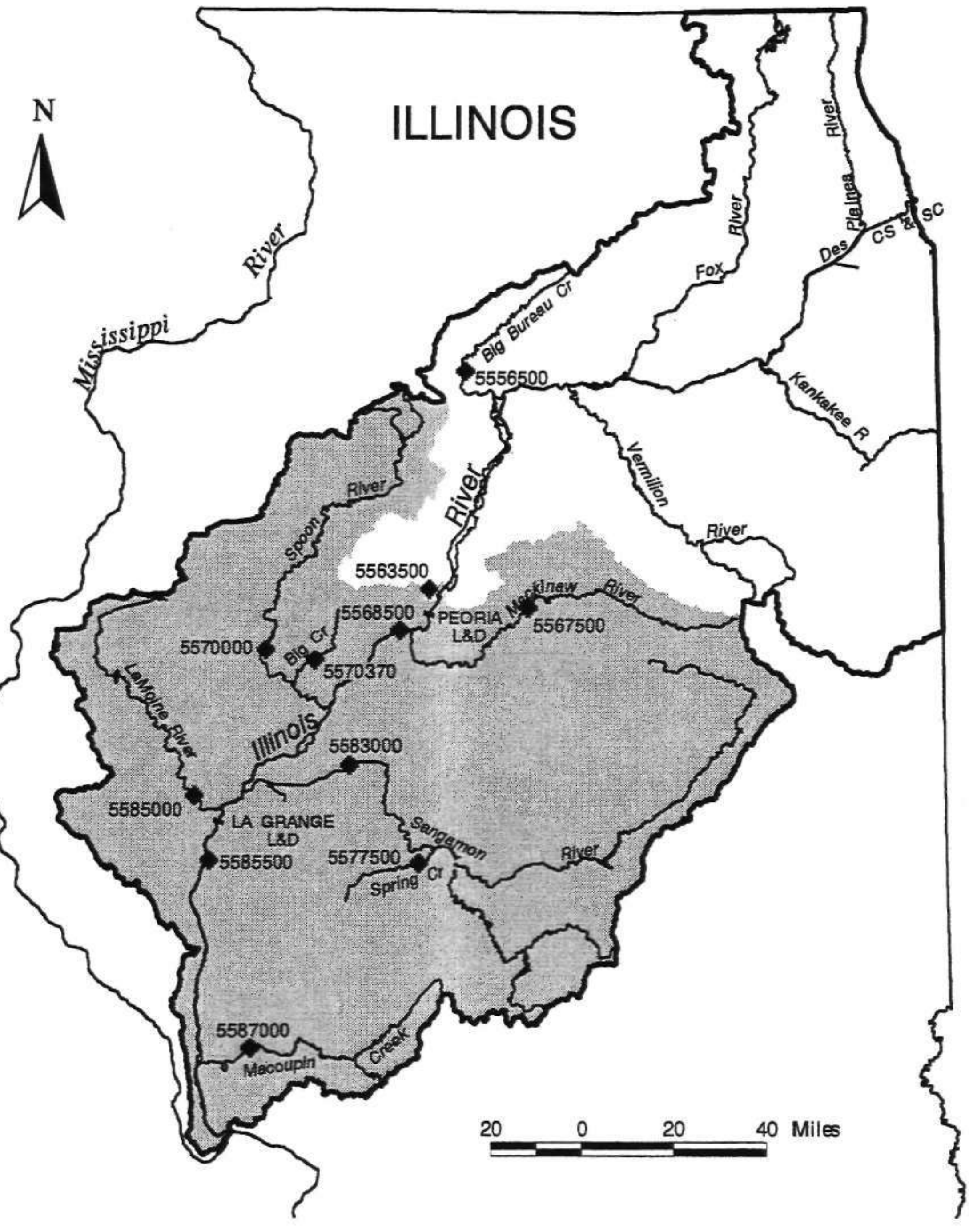

Figure 1. Drainage area of the lower Illinois River (shaded), major tributaries, gaging stations, La Grange Pool (Peoria Lock and Dam to La Grange Lock and Dam), and Alton Pool (La Grange Lock and Dam to confluence with the Mississippi River)

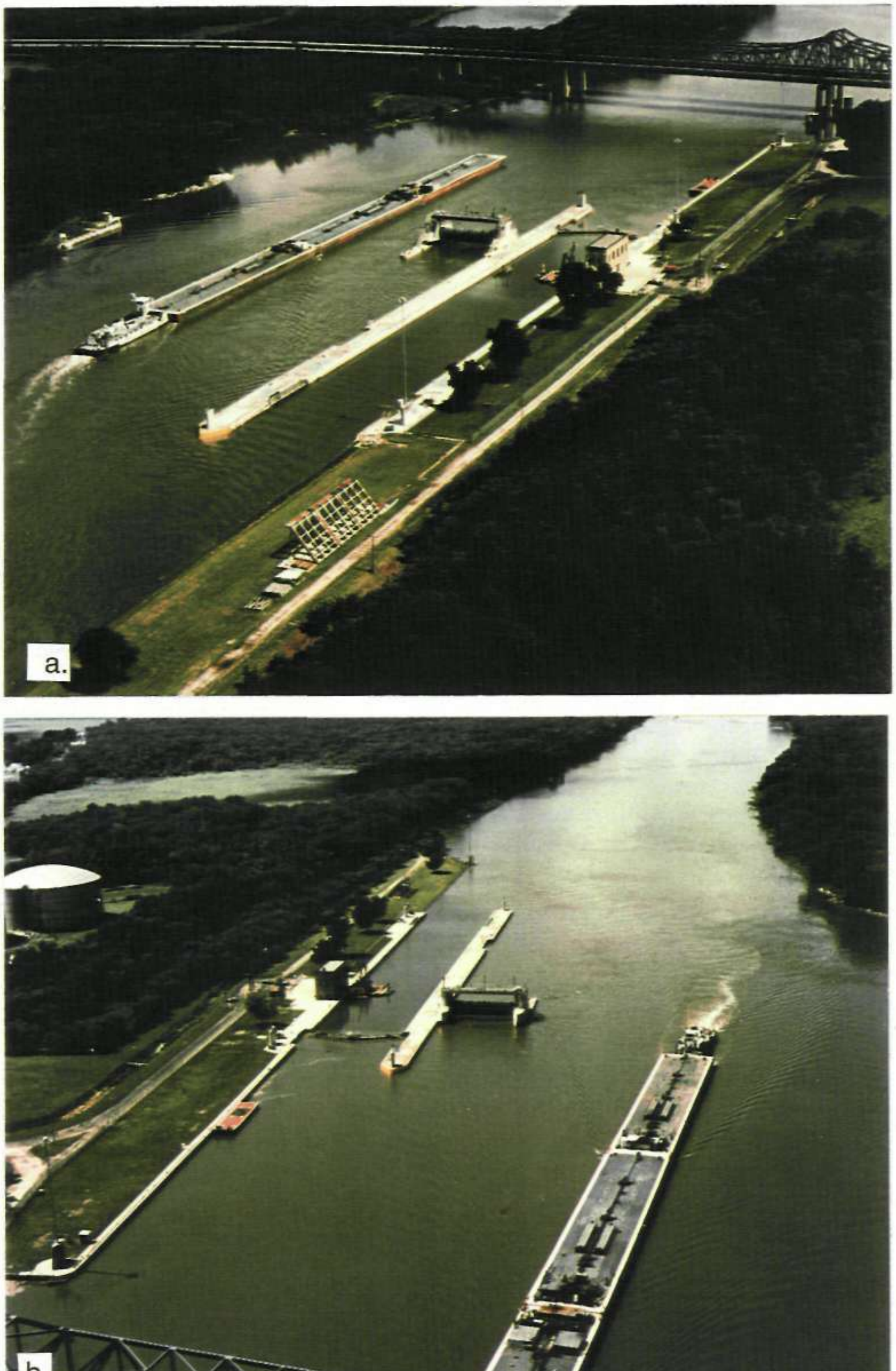

Figure 2. Aerial view of the Peoria Lock and Dam during flood stages when the wicket gates are lowered: (a) looking upstream, the lock and adjacent taintor gate are at the center of the photo and the Peoria Bridge is near the top of the photo and (b) looking downstream from Peoria Bridge

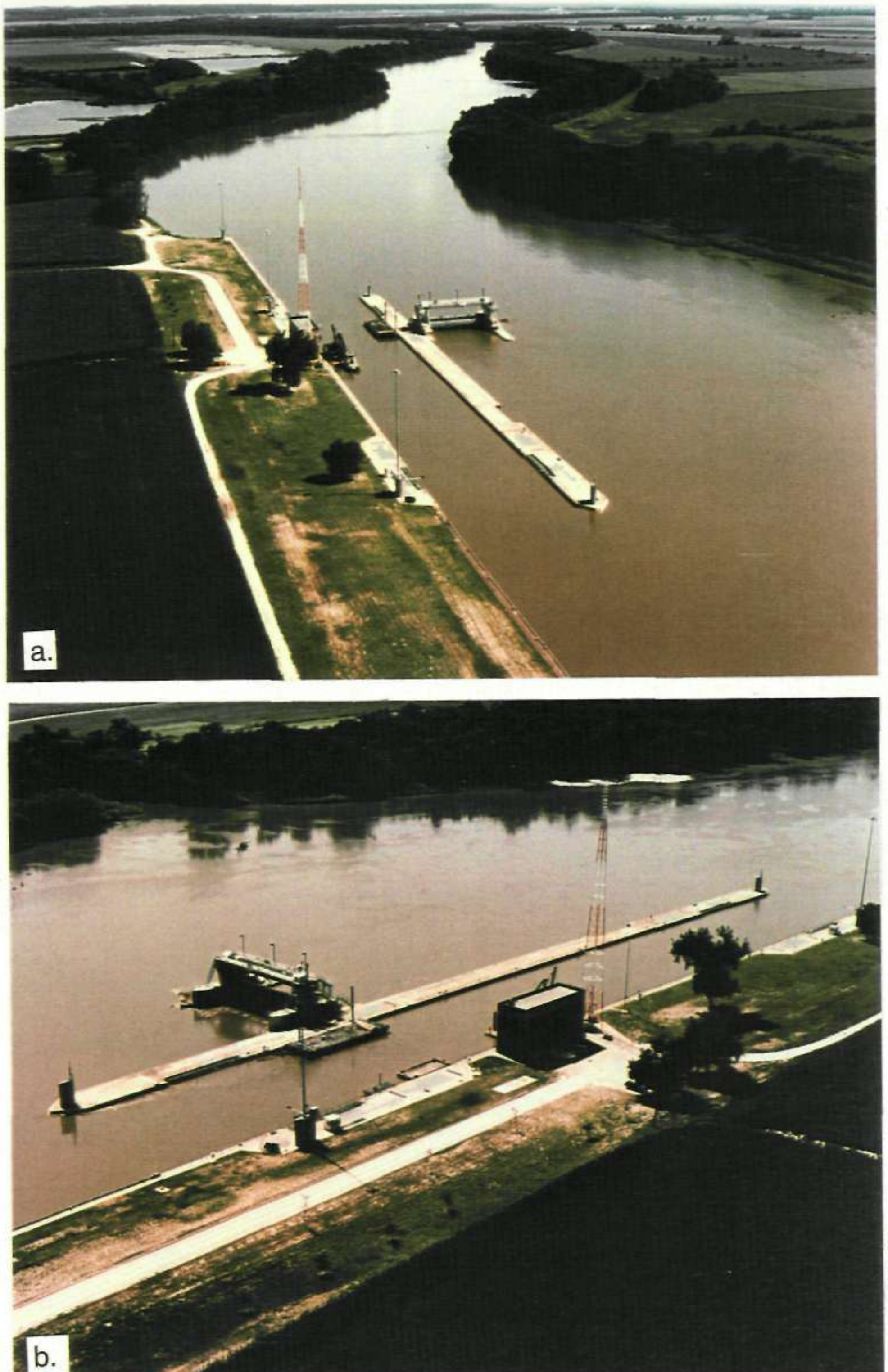

Figure 3. Aerial view of the La Grange Lock and Dam under flooding conditions: (a) looking upstream and (b) looking downstream, the lock and taintor gate are on the west bank and the undersluice gates are impounded

outlet ports containing butterfly valves is located west of the wicket gates. An earth dike extends from the regulating dam to the west bank.

The La Grange lock on the west bank of the Illinois River has the same dimensions as the Peoria lock. The L&D has a taintor gate and a dam that consists of wicket gates similar to the ones at Peoria. The dam is 436 feet long from the taintor gate to the regulating dam on the west bank. The concrete regulating dam is 136 feet long and has 12 equally spaced butterfly valves 6 feet × 6 feet. A 390-foot earth dam connects the regulating dam to the west bank of the river.

Figure 4 shows the levee and drainage districts (LDDs) located downstream of Peoria L&D. There are 13 active LDDs in each of the La Grange and Alton pools. Table 1 contains the list of the LDDs, the riverbank on which each levee is located, the riverfront extent of the levee, the area within each LDD, the year the district was organized, and other pertinent information.

In the upper reach of the study area, the four major tributaries that drain into the La Grange Pool are the Mackinaw, Spoon, Sangamon, and La Moine Rivers (Figure 1). In the Alton Pool, the only major tributary is Macoupin Creek. The drainage area of the segment of the Illinois River watershed that drains into La Grange Pool is 11,094 square miles (sq mi), and the watershed area for the Alton Pool up to the Mississippi River confluence is 3,258 sq mi. The combined drainage areas of the five major tributaries constitute about 90 percent of the total drainage area. The drainage area of the Sangamon River watershed alone is about 50 percent of the entire lower Illinois River watershed area (below Peoria L&D). Table 2 shows the drainage areas of some of the tributaries (available from the existing records) flowing into the lower Illinois River.

There are several gages located on the drainage area of the lower Illinois River (Figure 1). On the Illinois River, there are five gages in the La Grange Pool and six gages in the Alton Pool. Some of the gages are maintained by the U.S. Geological Survey (USGS) and the stage-only stations are under the jurisdiction of the USCOE. Table 3 lists the river mile locations of the gaging stations, the corresponding upstream drainage areas, gage elevations, and maximum flood elevations during the specified periods of records. Only the records after 1940 are used in the analyses.

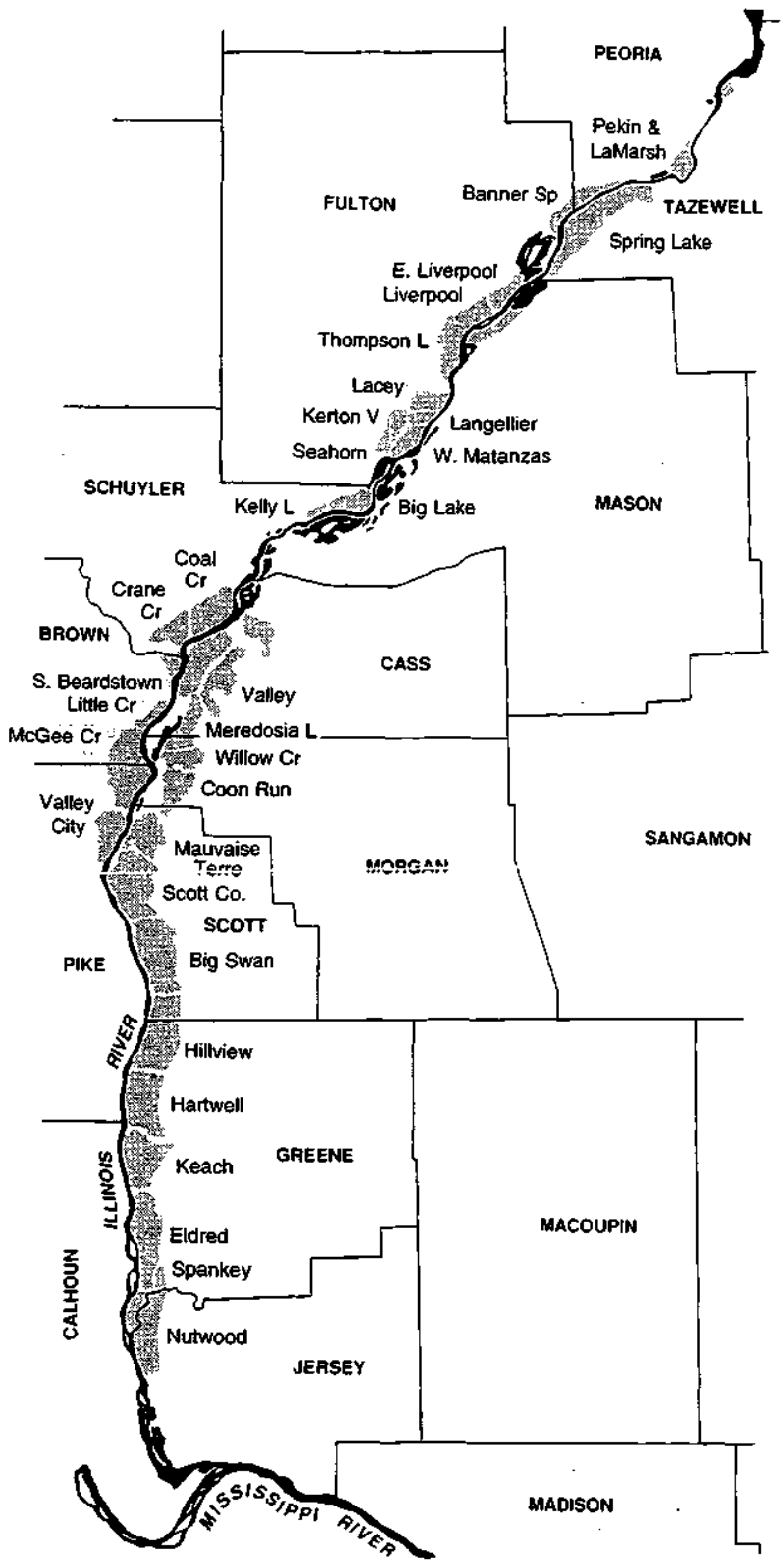

Figure 4. Levee and Drainage Districts in the La Grange and Alton Pools of the lower Illinois River

Table 1. Active Drainage and Levee Districts along the Illinois River

Levee district	*Bank position*	*Approximate riverfront limits of levee (mile)*	*Drainage district (acreage)*	*Design crown elevation (feet-msl)*	*Year organized*	*Remarks*
Peoria County						
Pekin & Lamarsh	Right	155.3-149.7	2,722	458.7-458.1	1888	
Tazewell County						
Spring Lake	Left	147.5-134.0	13,100	459.0-456.0	1903	
Fulton County						
Banner Special	Right	145.5-138.0	3,957	457.0-456.0	1910	Also Peoria County
East Liverpool	Right	131.7-128.5	2,885	455.4-455.0	1916-23	
Liverpool	Right	127.0-126.3	2,885	455.0	1916-23	
Thompson Lake	Right	126.0-121.0	5,498	453.0-451.0	1918-21	
Lacey, Langellier, W. Matanzas & Kerton Valley	Right	119.5-112.0	7,800	455.3-454.6	1893,1913,1917	
Schuyler County						
Big Lake	Right	108.3-102.8	3,401	451.7-451.2	1905	
Kelly Lake	Right	102.6-100.4	1,045	456.0-455.0	1916	
Coal Creek	Right	92.0-85.0	6,400	454.3	1895	
Crane Creek	Right	85.0-83.6	5,417	451.0-450.0	1908	
Cass County						
S. Beardstown & Valley City	Left	87.5-79.0	11,600	455.2-454.4	1913	Also Brown County
Meredosia Lake & Willow	Left	78.0-73.0	8,089	449.0	1893,1904	Also Morgan County

Table 1. Active Drainage and Levee Districts along the Illinois River (Concluded)

Levee district	*Bank position*	*Approximate riverfront limits of levee (mile)*	*Drainage district (acreage)*	*Design crown elevation (feet-msl)*	*Year organized*	*Remarks*
Brown County						
Little Creek	Right	78.8-75.2	1,611		1893	
McGee Creek	Right	75.2-67.3	10,800	444.0	1905	Also Pike County
Coon Run	Left	72.7-67.1	4,511		1902	Also Morgan County
Pike County						
Valley City	Right	66.3-63.0	4,750		1920	
Scott County						
Mauvaise Terre	Left	67.0-63.0	3,961		1902	Includes Roberson Private Levee District
Scott County	Left	63.0-56.8	10,245	446.0	1909	
Big Swan	Left	56.7-50.2	12,055	443.0	1903	
Greene County						
Hillview	Left	50.0-43.2	12,396	443.0	1906	Also Scott County
Hartwell	Left	43.0-38.3	8,696	441.0	1906	
Keach	Left	38.0-32.7	8,000	440.0	1922	
Eldred & Spanky	Left	32.2-23.6	9,300	440.0-436.0	1909,1917	
Jersey County						
Nutwood	Left	23.3-15.0	10,638	438.0	1907	

Note: Data were abstracted from the U.S. Army Corps of Engineers Publications USCOE (1987) and USCOE (1996).

Table 2. Illinois River Tributaries below Peoria Lock and Dam

Tributary	*Illinois River mile*	*Drainage area (sq mi)*
Otter Creek (L)	14.7	89.9
Macoupin Creek (L)	23.2	961.0
Panther Creek (R)	36.7	-
Apple Creek (L)	38.3	406.0
Hurricane Creek (L)	43.2	-
Sandy Creek (L)	50.0	166.0
Little Blue Creek (R)	54.1	-
Walnut Creek (L)	56.7	-
Big Blue Creek (R)	58.2	-
Mauvaise Terre Creek (L)	63.2	178.0
Coon Run (L)	66.9	-
McGee Creek (R)	67.0	444.0
Willow Creek (L)	71.2	-
Camp Creek (R)	75.7	-
Indian Creek (L)	78.7	286.0
Little Creek (R)	78.8	-
La Moine River (R)	83.5	1,350.0
Crane Creek (R)	84.9	40.2
Lost Creek (R)	89.0	16.5
Sugar Creek (R)	94.2	162.0
Sangamon River (L)	98.0	5,419.0
Elm Creek (R)	102.7	9.2
Wilson Creek (R)	108.3	13.3
Otter Creek (R)	111.8	126.0

Table 2. Illinois River Tributaries below Peoria Lock and Dam (Concluded)

Tributary	*Illinois River mile*	*Drainage area (sq mi)*
Spoon River (R)	120.5	1,855.0
Quiver Creek (L)	122.6	261.0
Big Sister Creek (R)	126.3	28.4
Buckheart Creek (R)	128.2	20.6
Duck Creek (R)	131.7	20.5
Copperas Creek (R)	137.4	127.0
Little LaMarsh Creek (R)	147.2	-
Mackinaw River (L)	147.7	1,136.0
LaMarsh Creek (R)	149.7	40.2
Lost Creek (L)	151.0	23.0
Lick Creek (L)	156.4	19.2

Note: Data were abstracted from Healy (1979) and USCOE (1987).
L = Tributary is on the left bank looking downstream; R = Tributary is on the right bank looking downstream.

Table 3. Gaging Stations on the Illinois River and Its Major Tributaries

Gage station location	*USCOE station #*	*USGS Station #*	*Distance from river mouth (mile)*	*Drainage area (sqmi)*	*Record max. daily water surface elevation (feet-msl*	*and date)*	*Zero gage elev. (feet-msl)*	*Period of record*
Mississippi R. at Grafton	0218A	05587450	-0.2	28,906	441.80	(8/1/93)	403.79	1879-1993
Illinois R. at Hardin	IH21	05587060	21.5	28,690	442.30	(8/3/93)	400.00	1932-1993
Illinois R. at Pearl	IP43	05586450	43.2	27,136	442.75	(8/3/93)		1878-1993
Illinois R. at Florence	IF56		56.0	26,919	443.60	(8/1/93)		1930-1993
Illinois R. at Valley City	IVC61	05586100	61.3	26,564	444.91	(5/26/43)	414.10	1938-1993
Illinois R. at Meredosia	IM70	05585500	71.3	26,028	446.69	(5/26/43)	418.00	1938-1993
Illinois R. at La Grange L&D (TW)		05585300	80.2	25,648	447.10	(5/26/43)	406.00	1937-1993
Illinois R. at Beardstown	11-0492-6	05584000	88.8	24,227	449.50	(5/26/43)	419.89	1940-1993
Illinois R at Havana	11-3940-4	05570520	119.6	18,299	451.00	(5/26/43)	424.40	1878-1993
Illinois R at Kingston Mines		05568500	144.4	15,819	454.00	(5/25/43)	428.00	1940-1993
Illinois R. at Peoria L&D (TW)		05560000	157.8	14,554	455.90	(5/24/43)	417.00	1940-1993
Spoon River at Seville		05570000	38.7	1,636			467.04	1914-1993
La Moine River at Ripley		05585000	12.3	1,293			431.10	1921-1993
Macoupin River at Kane		05587000	16.1	868			426.77	1921-1993
Mackinaw River near Green Valley		05568000	13.7	1,089			477.11	1921-1993
Sangamon River near Oakford		05583000	25.7	5,093			452.88	1910-1993

Notes:

TW = Tailwater
L&D = Lock and Dam
USCOE = U.S. Army Corps of Engineers
USGS = U.S. Geological Survey
sq mi = square mile
msl = mean sea level

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UNSTEADY FLOW MODEL STRUCTURE

To simulate the flow of water in a river channel, differential equations of mass and momentum are solved with a numerical method. Depending on the application, the partial differential equations could be used to describe steady and unsteady flow situations. Steady-state flow is based on the assumption that flow depth and discharge do not vary with time. It is well known that the flow in most rivers is unsteady. However, if the variation in the average discharge for several days following a storm and prior to the next storm is small, the flow can be considered steady flow for all practical purposes.

Due to the complexity in the numerical solution of the three-dimensional flow that is actually occurring in a river and the exorbitant cost of obtaining adequate field data, certain assumptions can be made to reduce the dimensions and hence the complexity of the hydrodynamic model. For instance, by assuming hydrostatic pressure conditions, the three-dimensional hydrodynamic equations will be reduced to those for a two-dimensional, vertically integrated model. Moreover, if the principal flow direction can be assumed to follow the centerline of the channel, the equations are further simplified and a one-dimensional set of equations is obtained. The latter assumption implies that the lateral variations in water depths and velocities are negligible. Examples of one-dimensional unsteady flow models are FLDWAV (Fread, 1985), FEQ (Franz, 1990), and UNET (HEC, 1993). These models are widely used for large river systems such as the Mississippi and Chesapeake Bay. Two- and three-dimensional models are mainly applied to short channel reaches to examine flow conditions around bridge piers, bends, and channel confluence. They have also been used to simulate tidal flow in lagoons and estuaries. Examples of two-dimensional models include FESWMS-2DH (Froehlich, 1989) and TABS-2 (Boss International, 1993).

The equations of conservation of mass and momentum for one-dimensional flow are based on the Saint-Venant derivations. The assumptions used in the derivation of these equations are:

- Constant average velocity at any section is perpendicular to the main flow direction.

- Negligible vertical acceleration implies that hydrostatic pressure predominates in the fluid, which means that the slope of the water surface profile varies gradually.
- Main flow direction along the stream centerline can be approximated with a straight line.
- Channel bottom slope is very small such that tan Ψ = sin Ψ, where Ψ is the angle of the channel bed to the horizontal surface.
- No scouring or deposition of sediment occurs on the channel bed.
- Channel roughness for steady flow is applicable for unsteady flow.
- Fluid is incompressible and has homogeneous density.

The mass and momentum equations can be expressed as:

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}+\frac{\partial S}{\partial t}=q_l \qquad (1)$$

and

$$\frac{\partial Q}{\partial t}+\frac{\partial\left(Q^2/A\right)}{\partial x}+gA\left(\frac{\partial h}{\partial x}+S_f\right)=q_l\ v_l \qquad \textbf{(2)}$$

where x is the distance along the channel, t is the time, Q is the flow, A is the cross-sectional area, h is the water depth, S is the storage volume per unit length in the direction of flow, S_f is the frictional slope, v_1 is the lateral inflow velocity, g is the gravitational acceleration, and q_1 is the lateral inflow per unit distance.

Most rivers during flood flow usually convey water within the channel banks and in the floodplains. The flow is essentially two-dimensional flow if depth averaging is assumed. The mass and momentum equations can be written for both the channel and floodplains as:

Channel:

$$\frac{\partial A_c}{\partial t} + \frac{\partial Q_c}{\partial x_c} = q_f \tag{3}$$

$$\frac{\partial Q_c}{\partial t} + \frac{\partial\left(Q_c^2/A_c\right)}{\partial x_c} + g A_c\left(\frac{\partial h}{\partial x_c} + S_{fc}\right) = M_c \tag{4}$$

Floodplain

$$\frac{\partial A_f}{\partial t} + \frac{\partial Q_f}{\partial x_f} + \frac{\partial S}{\partial t} = q_c + q_l \tag{5}$$

$$\frac{\partial Q_f}{\partial t} + \frac{\partial\left(Q_f^2/A_f\right)}{\partial x_f} + g A_f\left(\frac{\partial h}{\partial x_f} + S_{ff}\right) - \xi\, q_l\, v_l = M_f \tag{6}$$

M_c and M_f are the momentum flux exchanges per unit distance between the channel and floodplain, respectively; ξ is the fraction of the momentum entering the receiving stream; and the subscripts c and *f* represent the channel and floodplain. This momentum flux is the momentum of the flow passing through the channel section per unit time per unit distance along the channel. The water surface elevation is assumed to be the same for the channel and floodplain. Since the exchanges of mass between the channel and floodplain are equal, then $q_c\, \Delta x_c = q_f\, \Delta_{xf}$, where Δx_c and Δx_f are the lengths of the shoreline and bluff across which lateral inflow enters the channel and floodplain, respectively. Equations (3) and (5) can be manipulated to yield

$$\frac{\partial\left(A_f + \varphi\, \bar{A_c}\right)}{\partial t} + \frac{\partial Q}{\partial x_f} + \frac{\partial S}{\partial t} - q_l = 0 \tag{7}$$

where φ is equal to $\Delta x_c / \Delta x_f$ and $Q = Q_c + Q_f$.

Since the momentum exchanges between the channel and floodplain flows are also equal, i.e., $M_c \Delta x_c = -M_f \Delta x_f$, then equations (4) and (6) can be combined to yield the following expression:

$$\frac{\partial (Q_f + j Q_c)}{\partial t} + \frac{\partial}{\partial x_f}\left(\frac{Q_c^2}{A_c} + \frac{Q_f^2}{A_f}\right) + g\left(A_c + A_f\right)\frac{\partial h}{\partial x_f} + j\, g\, A_c\, S_{fc}$$
$$+ g\, A_f\, S_{ff} - x\, q_l\, v_l = 0$$

(8)

If an equivalent frictional force is defined as

$$g\, A\, S_f\, \Delta x_e = g\, A_c\, S_{fc}\, \Delta x_c + g\, A_f\, S_{ff}\, \Delta x_f \qquad (9)$$

and a velocity distribution factor, β, as

$$\beta = \frac{Q_c^2/A_c + Q_f^2/A_f}{Q^2/A} \qquad (10)$$

then equation (8) can be expressed in simplified form as

$$\frac{\partial\left(Q_f + \varphi\, Q_c\right)}{\partial t} + \frac{\partial\left(\beta\, Q^2/A\right)}{\partial x_f} + g\, A\left(\frac{\partial h}{\partial x_f} + s_f\,\frac{\Delta x_e}{\Delta x_f}\right) - \xi\, q_l\, v_l = 0 \qquad (11)$$

where Δx_e is the equivalent flow path, S_f is the frictional slope for the entire cross section, and $A = A_c + A_f$ is the total cross-sectional area.

Method of Solution

Equations (7) and (11) can be solved by finite difference or finite element methods. The finite element method is usually very cumbersome and may produce unstable solutions for hyperbolic equations unless upwinding terms are added to the numerical scheme (Akanbi and Katopodes, 1988). Finite difference methods, on the other hand, are easier to develop and have been widely applied to fluid flow problems. Explicit finite difference schemes are used to solve for the unknown nodal variables at a new time step by using the previous time step solutions. However, for implicit schemes, the nodal solutions are solved simultaneously for all nodes at the new time step. An initial guess of the solution is made at the beginning of a new time step, and an iterative procedure is then used to update the variables until a specified error criterion is satisfied. Examples of finite difference schemes that have been applied to fluid flow problems include the Method of Characteristics, Leap Frog, Lax-Wendroff, and Four-Point or Box schemes.

Equations (7) and (11) have been solved with the implicit four-point scheme. The weighted four-point implicit scheme was first developed by Preissman (1960). The discretization of the depth of flow, *h*, and its derivatives are given by:

$$h(x,t) = \frac{\theta}{2}\left(h_{j+1}^{n+1} + h_j^{n+1}\right) + \frac{1-\theta}{2}\left(h_{j+1}^{n} + h_j^{n}\right) \tag{12}$$

$$\frac{\partial h}{\partial t} = \frac{h_{j+1}^{n+1} - h_{j+1}^{n} + h_j^{n+1} - h_j^{n}}{2\Delta t} \tag{13}$$

$$\frac{\partial h}{\partial x} = \theta\,\frac{h_{j+1}^{n+1} - h_j^{n+1}}{\Delta x} + (1-\theta)\,\frac{h_{j+1}^{n} - h_j^{n}}{\Delta x} \tag{14}$$

where θ is the weighting factor, which varies between 0 and 1. When 9 is zero, an explicit scheme is obtained. The scheme is fully implicit when 9 is equal to 1. When the derivatives in equations (7) and (11) are replaced with the finite difference schemes (equations 12-14), and by grouping the unknown quantities on the left-hand side of the resulting expressions, the following linear algebraic equations are obtained:

$$A_j \, \Delta Q_j + B_j \, \Delta h_j + C_{j+1} \, \Delta Q_{j+1} + D_{j+1} \, \Delta h_{j+1} = E_j \tag{15}$$

$$A'_j \, \Delta Q_j + B'_j \, \Delta h_j + C'_{j+1} \, \Delta Q_{j+1} + D'_{j+1} \, \Delta h_{j+1} = E'_j \tag{16}$$

The matrix coefficients $A_j, B_j, C_j, D_j, E_j, A_j, B_j, C_j, D_j,$ *and* E_j are defined in the appendix. Δh and ΔQ are the changes in h and Q between two consecutive time steps. Equations (15) and (16) are combined to form a single matrix of equations which is solved with the Gaussian Elimination Procedure (Bathe and Wilson, 1976; HEC, 1993). The Gaussian Elimination technique converts the sparse matrix into a triangular system which is easier to solve.

Boundary Conditions

In the UNET model, a simple reach of a river will be subdivided into N-l finite difference cells, which are bordered by N computational nodes. By writing continuity and momentum equations for each cell, a total of 2N-2 equations will be formed. However, since there are 2N unknown Δh and ΔQ at the N nodes, two additional equations are needed in order to determine all unknown quantities. These extra equations are provided by the boundary conditions at the upstream and downstream ends of the reach. The UNET model simulates subcritical flow, the prevalent condition in most natural streams. However, the model allows the user to specify supercritical flow condition at the upstream boundary.

The upstream boundary condition can be specified as a stage or water surface elevation hydrograph; or as a discharge hydrograph. The downstream boundary condition can be a stage hydrograph, a discharge hydrograph, or a known relation between stage and discharge such as a rating curve. The specification of any one of these boundary conditions will be satisfactory. If the stage is used, then the discharge, Q, is computed in the solution of the Saint-Venant equations. Similarly, if the discharge is specified, then the stage, *h*, is computed.

Any combination of upstream-downstream boundary conditions can be specified. It has been observed that when discharge hydrographs are prescribed for both upstream and downstream boundary conditions, any error in the initial conditions will be magnified as the solution progresses. However, the errors are usually damped out after a few time steps for the other combinations of boundary conditions.

In a typical application of the UNET model for flood flow prediction, the upstream boundary condition will usually be a discharge or stage hydrograph while at the downstream boundary a stage or stage-discharge rating relation is specified. A stage hydrograph is usually prescribed at the downstream boundary when the boundary is influenced by tidal actions.

Dendritic River System

In a dendritic river system, the main stem is subdivided into several subreaches such that each of the subreaches represents a segment of the river between the confluence of two adjacent tributaries. For a tributary that experiences significant backwaters due to floods on the main river and for which cross-sectional data are available, the tributary can be divided into several cells, the total number of cells depending on the degree of variability of the cross-sectional geometry along the river. For tributaries without available cross-sectional geometry data or not affected by backwater flow, the tributary flow can be input into the model as a point or a uniformly distributed inflow along the banks of the main river stem.

Figure 5 is an example of a river system that has two tributaries. The main river stem is divided into three reaches comprising the river segment between the upstream end and the mouth of tributary #1, the river segment between the two tributaries, and the river segment from the mouth of tributary #2 to the downstream end. There will be a total of five reaches if the two tributaries are included. Each of the reaches has been divided into finite difference cells bounded by nodes. The nodes correspond to the locations where

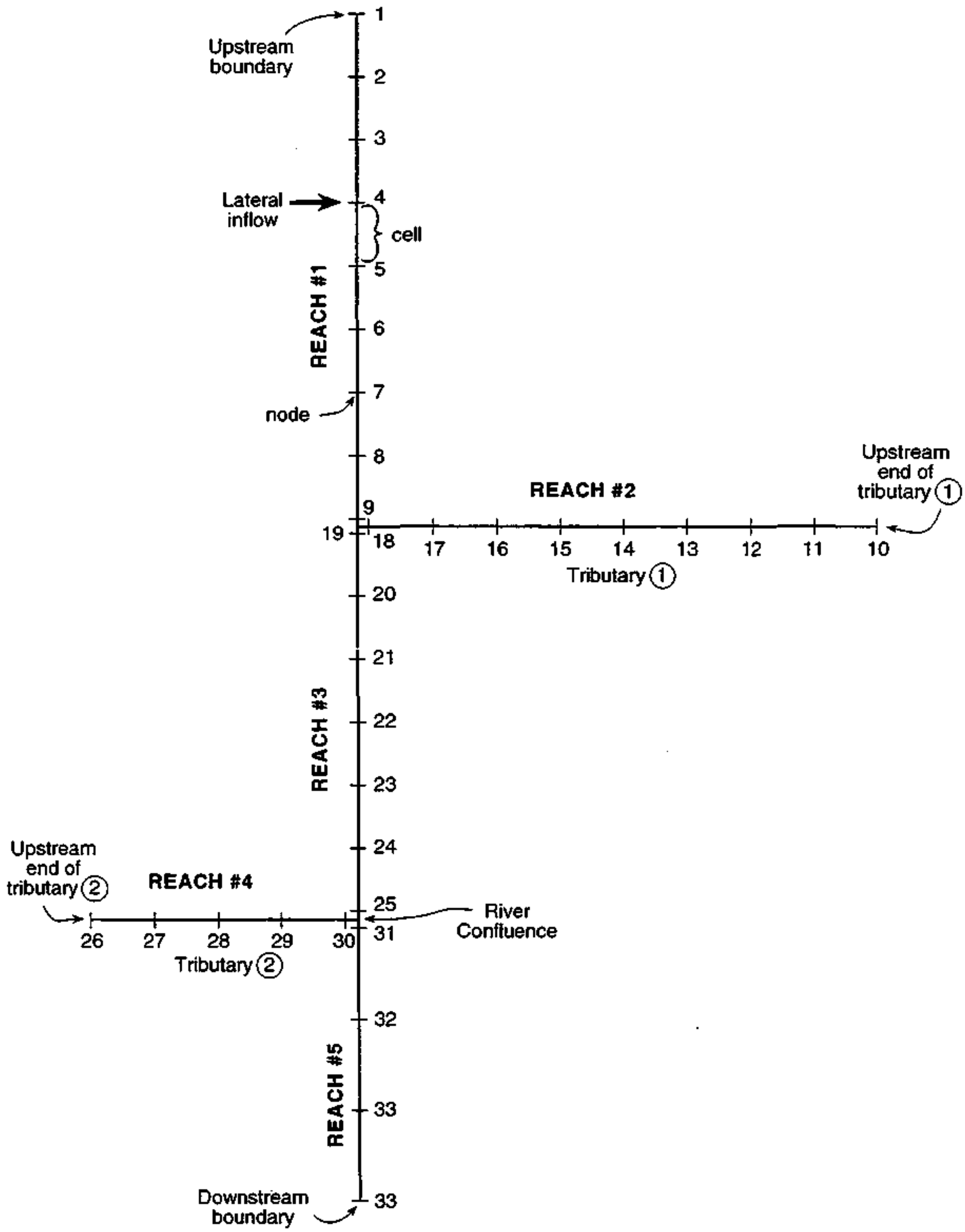


Figure 5. A two-tributary river system is divided into subreaches and finite difference cells

cross-sectional geometries have been measured or estimated. The nodes are numbered starting at the upstream end of a reach and increase sequentially towards the downstream end. The node numbering commences at the upstream end of reach 1, the uppermost reach of the main stem, to the downstream end of the reach. The numbering continues from the downstream end of reach 1 to the upstream boundary of reach 2 (tributary #1). This node numbering procedure is repeated for reaches 3, 4, and 5. The node numbers and the total number of cells for the five reaches in this example are given below:

Node Numbers

Reach	Upstream/Downstream	Total Number of Cells
1	1-9	8
2	10-18	8
3	19-25	6
4	26-30	4
5	31-34	3

Lateral Inflow

UNET incorporates tributary inflows through the lateral inflow term, q_l, in equations (7) and (11). q_l is specified as either a point inflow hydrograph or a uniformly distributed hydrograph between any two specified cross sections. For point inflow, q_l will represent the total flow entering the reach. The effect of the lateral inflow will be observed at the next downstream cross section.

Channel Cross Sections

A channel cross section can be represented by regular or irregular geometry. The cross sections are usually taken at gaging station locations, at locations where the changes in cross-sectional geometry significantly affect the flow, at the confluence of tributaries, and around hydraulic structures such as bridge piers, culverts, weirs, and locks and dams. The surveyed sections are input into the model as pairs of elevation and distance from a predetermined point on the left bank of the stream looking in the downstream direction.

The UNET model also allows the demarcation of active flow areas as well as dead storage areas in a cross section. This enables the model to adequately represent bank encroachment, embankments, bridge piers and openings, levees, and floodways.

Roughness Coefficient

The Manning's roughness coefficient, n, is used to describe the resistance to flow due to bed forms, vegetation, bends, and eddies. Roughness coefficients are specified for the bank-full channel area, and for the left and right overbank areas of each cross section. The model computes area-weighted average roughness values for the left and right overbank areas in order to determine the roughness value for the floodplain component of the flow. Also, the model allows conveyance, which is inversely proportional to the Manning's roughness, to be varied with discharge.

Initial Condition

Initial flow distributions are required for each of the reaches. For instance, if the initial flow condition is assumed to be steady state in the example in Figure 5, five flow distributions will be specified for the river system. Required model input also includes the initial water surface elevation for each of the storage areas if there are any in the river system.

LOWER ILLINOIS RIVER MODEL SETUP

The UNET model can simulate one-dimensional flow through single, dendritic, or looped systems of open channels. The model can simulate the interaction between channel and floodplain flows; levee failure and storage interactions; and flow through navigation dams, gated spillways, weir overflow structures, bridges and culverts, and pumped diversions.

The model solves the one-dimensional unsteady flow equations using a linearized implicit finite difference scheme. It requires a stage or discharge condition at the upstream boundaries of the main river and tributaries which have existing cross-sectional data. A stage or stage-discharge relation is prescribed at the downstream boundary of the main river. Other input requirements include tributary inflows, cross-sectional geometry, and hydraulic roughness parameters. The cross-sectional geometry and the stage or discharge boundary condition are prepared in two separate files. In addition, the model can read time-series data of stage and discharge from the Data Storage System (DSS) database that developed by the USCOE for the Hydraulic Engineering Center's (HEC) series of models. The output of the UNET model includes the time-series of stage and discharge at prescribed locations and plots of water surface elevation profiles. Figure 6 shows a flowchart representing the components of the UNET model and its relations with the DSS package.

Model Layout

As discussed in the previous chapter, the river system is usually set up as a series of interconnected reaches based on the number of tributaries and the location of their confluences with the main river. In the case of the lower reach of the Illinois River (Peoria Lock and Dam to Grafton), five gaged tributaries contribute significant flow to the Illinois River. Other tributaries to the Illinois River are ungaged. The Sangamon River is the only tributary that has existing cross-sectional data. Because of the lack of such data on the other major tributaries, the lower Illinois River has been set up as a system of three river reaches as shown in Figure 7. In this figure, Reach 1 is the segment of the Illinois River

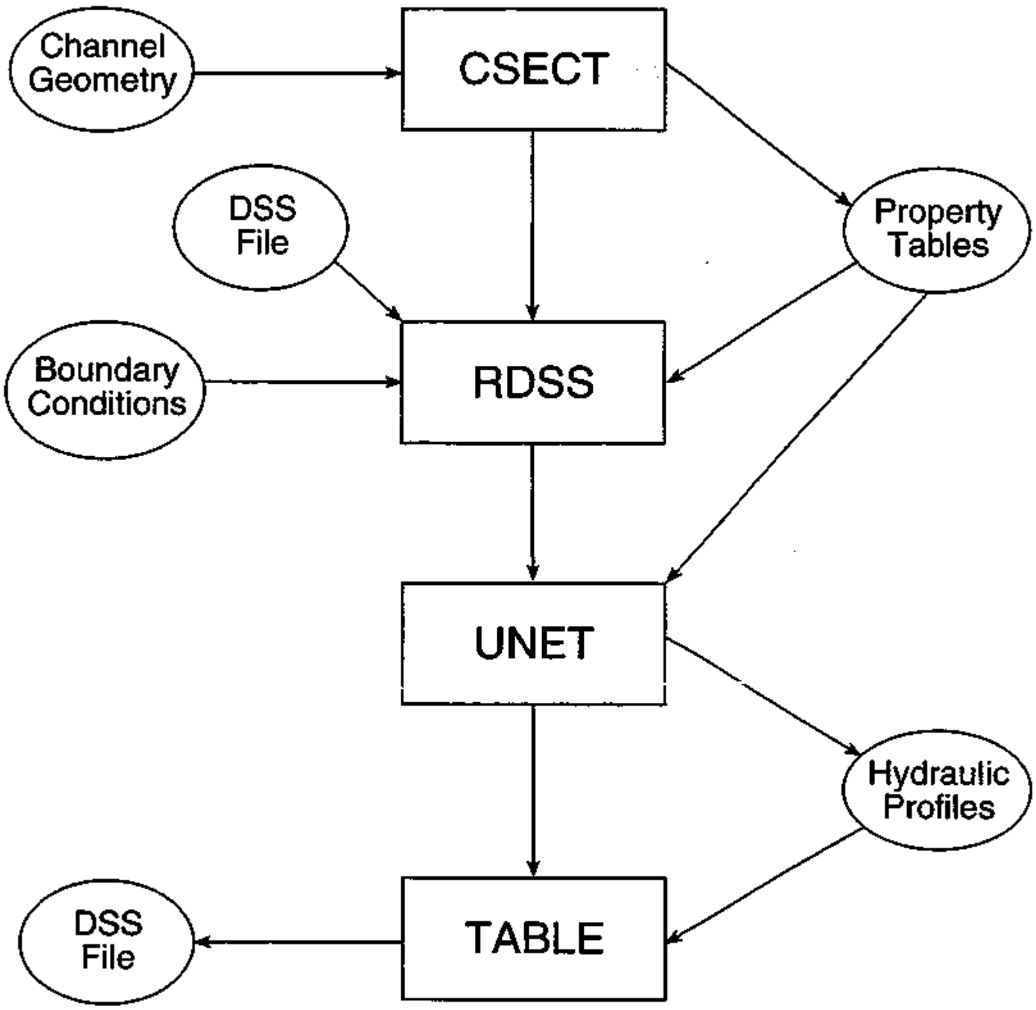


Figure 6. A flowchart represents components of the UNET model and its linkage with the Data Storage System database

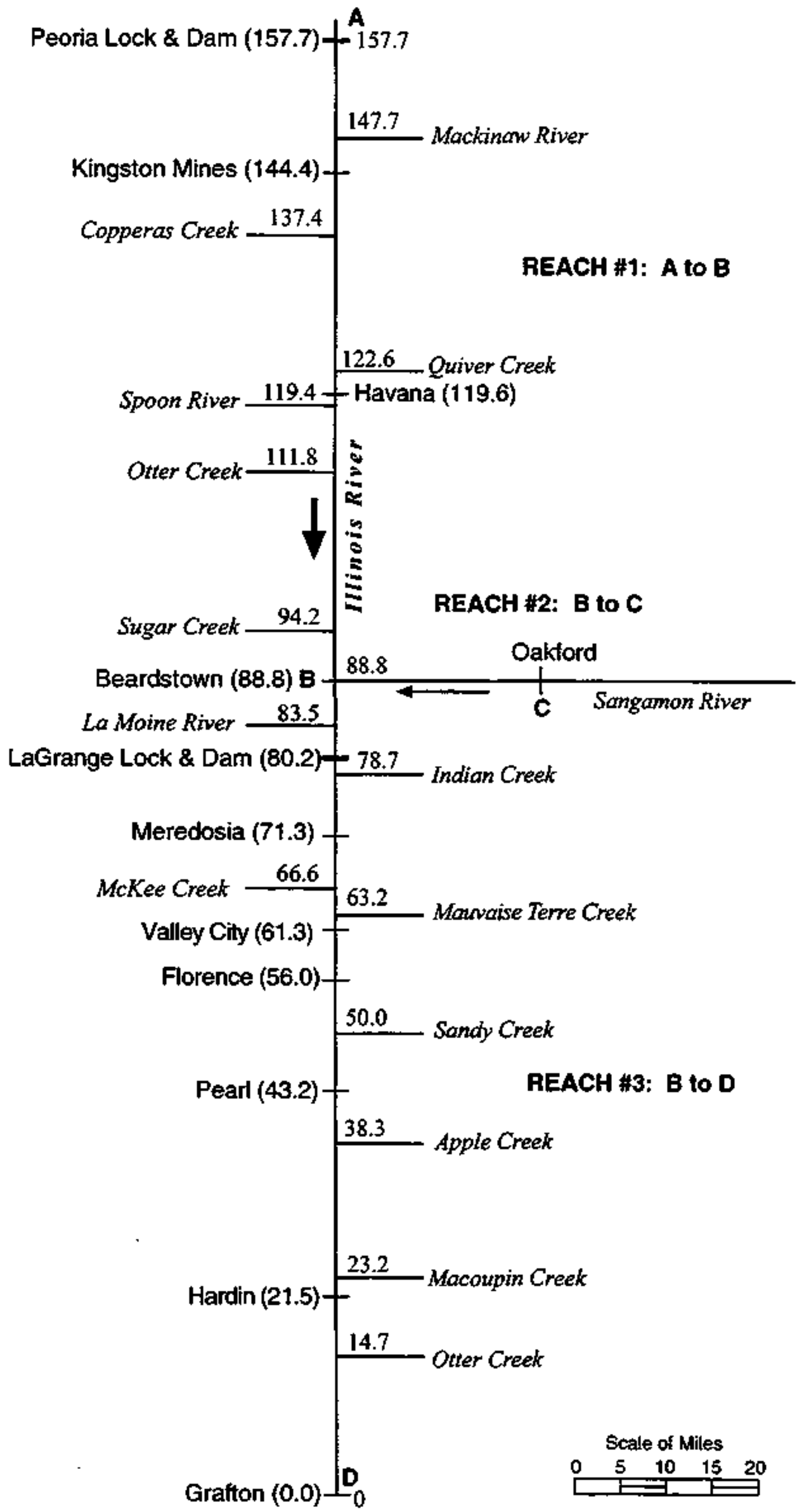


Figure 7. This model represents the lower Illinois River as a system of three river reaches and several lateral inflow areas

from Peoria Dam to the river section immediately upstream of the Sangamon River junction. Reach 2 is the segment of the Sangamon River from the gage at Oakford to the river mouth, and Reach 3 is the segment of the Illinois River from the Sangamon River junction to the Illinois River mouth at Grafton.

The Mackinaw and Spoon Rivers are the major tributaries in Reach 1 while the La Moine River and Macoupin Creek are the major tributaries in Reach 3. These tributaries and the smaller ones are taken as lateral inflow at their confluence with the Illinois River. The Sangamon River (downstream of Oakford) was assumed to have no point lateral inflow from its tributaries. Instead, the contributions from its tributaries were assumed to be uniformly distributed lateral inflow along the entire length of the Sangamon River. Figure 7 also shows the location of the discharge/stage gages on the Illinois River at Peoria L&D, Kingston Mines, Havana and Beardstown in Reach 1; and La Grange L&D, Meredosia, Valley City, Florence, Pearl, Hardin, and Grafton in Reach 3. It will be shown in the model calibration section below how stage and discharge hydrographs, generated by the model at these gage locations, are matched with historical records.

Cross-Sectional Data

The USCOE provided 412 surveyed cross sections from for the Peoria-Grafton section of the Illinois River. Thirty-three surveyed cross-sections were also obtained from the USCOE for the Sangamon River, starting from the gage at Oakford and extending to its confluence with the Illinois River. Figure 8 shows some typical cross sections at selected locations on the Illinois River.

Hydraulic structures such as levees, weirs, locks and dams, bridge piers, embankments, channel encroachments, and storage areas are given special considerations in the model. Within the selected study reach, the only major hydraulic structure, apart from bridge piers, is the La Grange L&D at river mile 80.2. The L&D has a dam section comprising wicket gates, a taintor gate structure, a regulating dam with butterfly valves, and a 390-foot earth dam.

There are 10 levees and drainage districts in Reach 1 and 15 LDDs in Reach 3. Table 4 lists data on the drainage area behind the levees, the average round elevation in

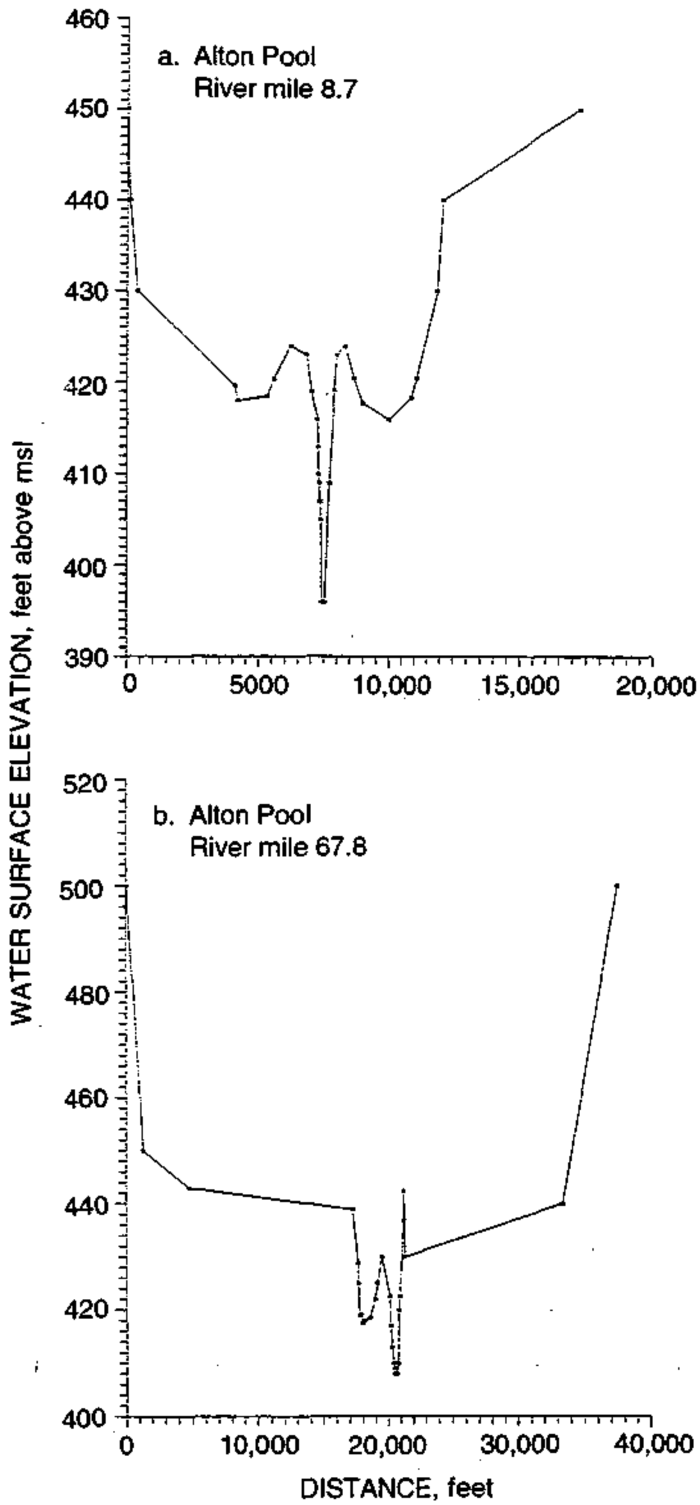


Figure 8. Selected cross sections of the lower Illinois River in the Alton Pool at River Miles (a) 8.7 and (b) 67.8, and in the La Grange Pool at River Miles (c) 81.6 and (d) 132.2

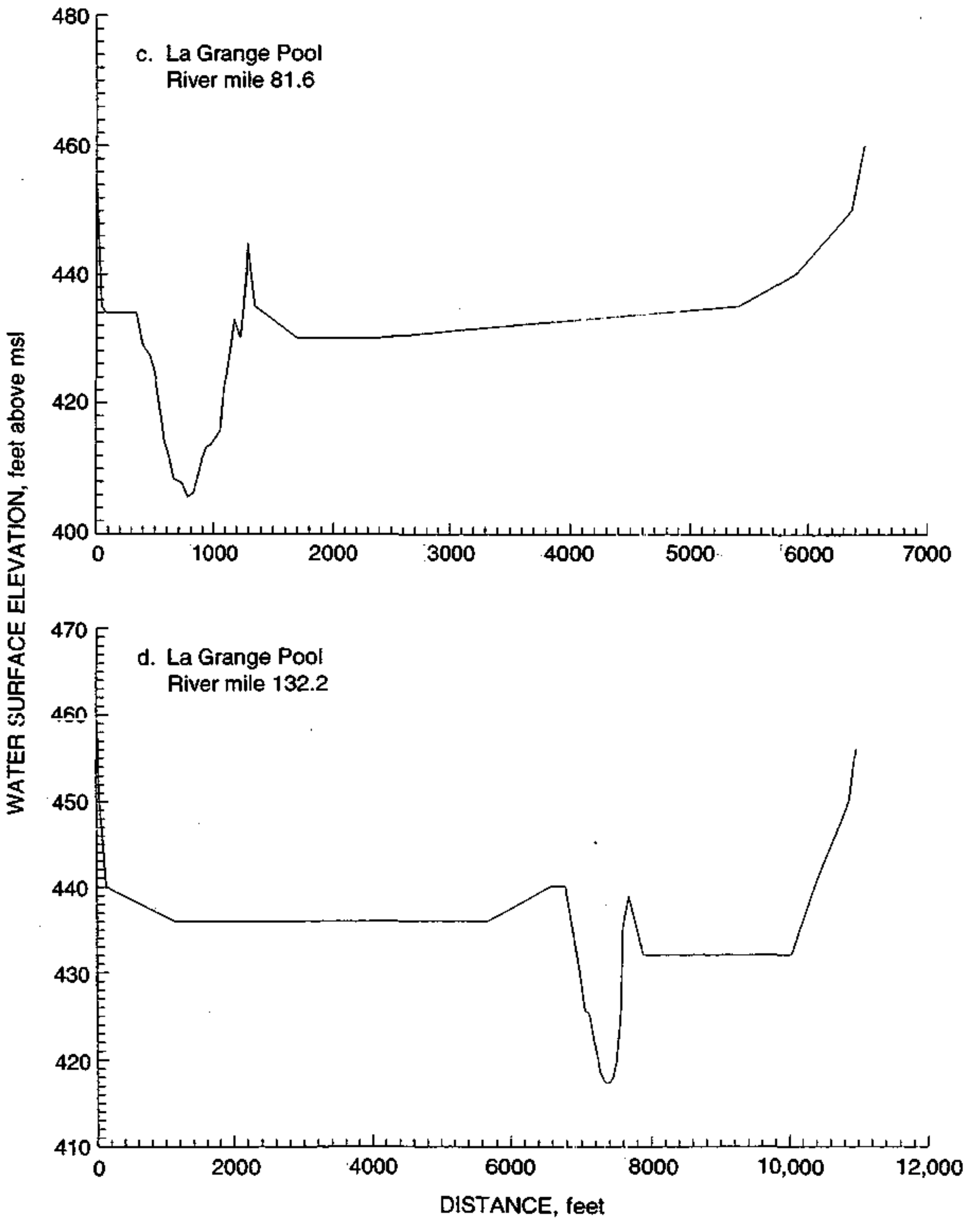


Figure 8. (concluded)

Table 4. Model Input Parameters for Levees and Drainage Districts

Levee district	*Protected area (acres)*	*Interior elevation (feet-msl)*	*River Mile*		*Breach elevation (feet-msl)*	*Cross section upstream of levee (mile)*	*Crown elevation*	
			Upstream	*Downstream*			*Upstream (feet-msl)*	*Downstream (feet-msl)*
Peoria L&D to Sangamon River								
Pekin& La Marsh	2,722	438.0	155.3	149.7	458.0	155.60	458.7	458.1
Spring Lake	13,100	430.0	147.5	134.0	455.0	151.20	459.0	456.0
Banner Special	3,957	440.0	145.5	138.0	455.6	145.70	457.0	456.0
East Liverpool	2,885	435.0	131.7	128.5	455.0	132.20	455.4	455.0
Liverpool	2,885	430.0	128.1	126.0	455.0	128.40	455.0	455.0
Thompson Lake	5,498	430.0	126.0	121.0	453.0	126.40	453.0	451.0
Lacey, Langellier, W. Mantaza & Kerton Valley	7,800	435.0	119.5	112.0	455.0	119.56	455.3	454.6
Big Lake	3,401	435.0	108.3	102.8	451.0	108.40	451.7	451.2
Kelly Lake	1,045	434.0	102.6	100.4	455.0	102.70	456.0	455.0
Sangamon River to Grafton								
Coal Creek	6,400	430.0	92.0	85.0	454.7	92.20	454.3	454.3
Crane Creek	5,417	430.0	85.0	83.6	450.0	85.50	451.0	450.0
S. Beardstown & Valley City	10,516	428.0	88.2	79.0	453.8	88.40	455.2	454.4
Little Creek	1,610	426.0	78.2	75.1	448.0	78.50	448.0	448.0
McGee Creek	12,400	430.0	75.0	67.1	445.5	75.50	445.5	445.5
Coon Run	6,162	438.0	72.7	67.1	448,5	72.80	448.3	448.5
Mauvaise Terre	6,626	440.0	65.8	63.4	446.0	66.00	446.0	446.0
Valley City	4,700	435.9	66.2	62.5	446.0	66.60	446.0	446.0
Scott County	12,700	433.6	63.1	56.7	446.0	63.30	446.0	446.0

Table 4. Model Input Parameters for Levee and Drainage Districts (Concluded)

Levee district	*Protected area (acres)*	*Interior elevation (feet-msl)*	*River Mile*		*Breach elevation (feet-msl)*	*Cross section upstream of levee (mile)*	*Crown elevation*	
			Upstream	*Downstream*			*Upstream (feet-msl)*	*Downstream (feet-msl)*
Big Swan	14,200	435.0	56.5	50.1	444.0	57.00	444.0	444.0
Hillview	13,700	427.9	50.0	43.2	443.5	50.05	443.5	443.5
Hartwell	9,300	426.9	43.1	38.2	442.5	43.17	442.5	442.5
Keach	9,700	429.9	38.0	32.8	441.5	38.20	441.5	441.5
Eldred & Spanky	9,800	427.4	32.4	23.8	441.5	32.70	441.5	441.5
Nutwood	10,600	428.3	23.6	15.1	440.0	24.26	440.0	440.0

Note: Data were abstracted from USCOE (1987, 1996).

the levee districts, upstream and downstream riverfront limits, and upstream and downstream crown elevations for Reaches 1 (Peoria-La Grange) and 3 (La Grange-Grafton).

The data input file includes cross sections of bridge crossings at river miles 71.31 (Main Street Bridge at Meredosia), 61.39 (Norfolk & Western Railway), 55.96 (U.S. Highways 36 and 54 or Florence Highway), and 21.65 (State Highway 100). However, the model did not include other bridge crossings at river miles 152.9 (Pekin Highway bridge), 152.3 (Chicago & Northwestern Railway), 119.5 (U.S. 136), 88.8 (Burlington Northern Railroad), 87.9 (Beardstown Highway or US 67 and State Route 100) and 43.2 (Illinois Central & Gulf Railroad or Alton Route) because the spans between adjacent piers were wide enough such that they do not significantly affect the river flow.

Flow and Stage Time-Series Data

The ability of the UNET model to use either the discharge or the stage upstream boundary condition, as previously discussed, is advantageous in situations where the upstream end of the reach is a stage-only station. This is the case at Peoria L&D where only stage records are available (1940 -1993) from the USCOE. The only other upstream boundary in the system is on the Sangamon River (Reach 2) at the site of the Oakford gage. This station has a long record of discharge (1910-1993). The downstream boundary is at Grafton (Reach 3), and the prescribed boundary condition is a time-series of stage for the model calibration and verification. A stage-discharge rating relation is used to simulate levee storage options.

A time-series of discharges is required for point lateral inflows from some of the tributaries listed in Table 2. The gaged tributaries have already been identified as the Mackinaw, Spoon, Sangamon, and La Moine Rivers, and Macoupin Creek. The lateral inflows from ungaged tributaries with drainage areas greater than 50 sq mi were assumed to be point discharges to the Illinois River. However, for all ungaged tributaries, within a particular segment, with drainage areas less than 50 sq mi, the flows were combined and assumed to be uniformly distributed as lateral inflows along that segment of the Illinois River.

Ungaged tributary flows can be estimated in several ways. For the present analysis, the ungaged tributary flows were estimated using records from a gage in a nearby watershed with similar hydrologic pattern. The discharge record at the selected gage will be scaled with the fraction of the area of an ungaged watershed to the area of a gaged watershed. Table 5 shows the river mile location, drainage area, drainage area ratio, and time lag for the ungaged tributaries and drainage areas. The time lag was estimated from the average velocity of each stream. The average velocity was obtained from the USGS records for the stations along the stream. The uniformly distributed lateral inflow was estimated by scaling the discharge records of the gaging station on a nearby watershed with the fraction of the "unbalanced" drainage area to the area of the hydrologically similar watershed. The "unbalanced" drainage areas are obtained from the calculations shown in the spreadsheet in Table 5.

The "unbalanced" watershed areas that contribute directly to the distributed lateral inflow are determined as the difference in the drainage area of a segment of the Illinois River bounded by two adjacent gages and the total sum of the drainage areas of tributaries that are discharging into the Illinois River between the two gages. For instance, the difference in drainage areas above Kingston Mines and Havana gages is 2410 sq mi (Table 5). However, the sum of the watershed areas for Copperas Creek, Quiver Creek, and Spoon River is 2024 sq mi. Therefore, the drainage area that is contributing to the distributed inflow is 386 sq mi.

Big Bureau, Big, Spring, Hadley, and Bay Creeks were selected as hydrologically similar streams for different segments of the lower Illinois River. The Big Bureau Creek record was used for the distributed lateral inflow between Peoria L&D and Kingston Mines, and for point inflow for Otter and Sugar Creeks. Big Creek data were used as point lateral inflow for Copperas and Quiver Creeks, and for the distributed inflow between Kingston Mines and Havana. Spring Creek records were used as point inflow for Indian Creek and as distributed inflow between Havana and Meredosia and between Ripley and the confluence of the La Moine River. Hadley Creek discharge records were used for McKee, Mauvaise Terre, and Sandy Creeks as point inflow, and Bay Creek data

Table 5. Estimation of Drainage Area Ratio for Ungaged Watersheds

Illinois River gaging station	*Tributaries*	*Illinois River Mile*	*Drainage area (sqmi)*	*Representative stream for ungaged tributary*	*Drainage area ratio*	*Lag parameter (day)*
Peoria L&D, TW		157.7	14,554			
	Mackinaw River	147.7	1,136			1
	U D (Peoria-K.M.)	---	129	Big Bureau Cr.	0.66	1
Kingston Mines		144.4	15,819			
	Copperas Creek	137.4	127	Big Creek	3.15	1
	Quiver Creek	122.6	261	Big Creek	6.48	1
	Spoon River	120.5	1,855			1
	U D (K.M.-Havana)	---	167	Big Creek	4.14	1
Havana		119.6	18,229			
	Otter Creek	111.8	126	Big Bureau Cr.	0.64	1
	Sugar Creek	94.2	162	Big Bureau	0.83	1
	Sangamon River	88.8	5,419			1
	La Moine River	83.5	1,350			1
	U D (Havana-La Grange)	---	362	Spring Creek	3.38	1
La Grange TW		80.2	25,648			
	Indian Creek	78.7	286	Spring Creek	2.67	1
	U D (La Grange-Meredosia)	---	94	Spring Creek	0.88	1
Meredosia		71.3	26,028			
	McGee Creek	67.0	444	Hadley Creek	6.11	1
	Mauvaise Terre Creek	63.2	178	Hadley Creek	2.44	1
	Sandy Creek	50.0	166	Hadley Creek	2.28	1
	Apple Creek	38.3	406	Bay Creek	10.3	1

Table 5. Estimation of Drainage Area Ratio for Ungaged Watersheds (Concluded)

Illinois River gaging station	*Tributaries*	*Illinois River Mile*	*Drainage area (sqmi)*	*Representative stream for ungaged tributary*	*Drainage area ratio*	*Lag parameter (day)*
	McCoupin Creek	23.2	961			1
	Otter Creek	14.7	90	Bay Creek	2.28	1
	U D (Meredosia-Grafton)	---	633	Bay Creek	16.04	1
Illinois River - Mississippi River Confluence (0.2 mi upstream of Grafton)		0.0	28,906			

Notes: TW - Tailwater
U D - Uniformly Distributed Lateral Inflow
L&D - Lock and Dam

were applied to Apple and Otter Creeks. Hadley Creek were also used as distributed inflow between Meredosia and Grafton.

Channel Roughness

The Manning roughness coefficients for the cross sections on the Illinois and Sangamon Rivers were estimated from field reconnaissance surveys undertaken by the USCOE. The roughness coefficients will be updated by calibrating the model to historical flood data. In the UNET model, the Manning roughness coefficients are adjusted by modifying the channel conveyance. The channel conveyance is the capacity of the channel to transport water. Since the roughness coefficient is inversely proportional to the conveyance, the roughness coefficient will increase as the conveyance is reduced and vice versa. The model handles the change in conveyance by introducing a factor that is used to multiply the conveyance. When the adjustment factor is less than unity, the conveyance is reduced and the roughness coefficient is increased. If the factor is greater than one, the conveyance is increased while the value of the roughness coefficient is reduced. A value of one for the factor leaves both the conveyance and roughness coefficient unchanged.

The model provides two input options for modifying the channel conveyance. In the first option, a pair of factors for the channel and overbank areas of the cross sections is input into the model. Alternatively, if the roughness coefficient is observed to change with stage and discharge, a table of discharge versus adjustment factor can be input to the program.

Details of the procedure for calibration of the UNET model for the lower Illinois River are presented in the next chapter.

Initial Flow Condition

The initial flow condition for each of the model reaches can be specified in the time-series input file for the UNET model. The initial flow condition is specified in the direction of the backwater flow, which is from downstream to upstream for subcritical flow. If the initial condition is not specified, the model will assume steady-state subcritical flow and generate initial discharge and water surface elevation for each of the model reaches. No initial conditions were specified for the simulations presented in the next chapter.

If storage areas are present in the system, the initial water surface elevation is prescribed for each storage area. In the lower Illinois River, only one storage area was included in the model. The storage volume and the initial water surface elevation were included in the time-series data file.

MODEL CALIBRATION AND VERIFICATION

The UNET model can be applied to predict existing conditions, historical floods, statistically significant flood events, and combinations of feasible scenarios. In order to ensure that the simulation results are reliable and closely represent actual events, the model has to be applied initially to simulate selected historical floods in the river system. A level of error is pre-selected such that when the weighted sum of the differences between computed and observed water surface elevations (WSEs) are below this level, the computed WSE is taken as a true representation of the observed event. The fitting of the computed WSE hydrographs to historical events is called model calibration. The WSE and the flow computed by the model are adjusted to fit closely to observed data by gradually varying the channel roughness coefficient until the difference between computed and observed WSE is below the specified level of error. The Manning roughness coefficient is expressed in the model in terms of the channel conveyance. Initial roughness coefficient values are supplied to the model through the cross-section input file and are then converted to conveyances. The conveyances are updated during the model calibration by multiplying them by an adjustment factor that varies between 0 and 1. This factor is included in the time-series input file and is varied until the error in the computed WSE is below the tolerance.

Since subcritical flow is assumed in the UNET model, the calibration of the model will start from the downstream end of the study reach and progress in the upstream direction. The first calibrated section is between the downstream boundary and the next upstream gage, the river section between Grafton and Hardin in the Peoria-Grafton section of the Illinois River. If stage or water surface elevation is prescribed as the downstream boundary condition (at Grafton), the conveyance in this river segment will be adjusted until the computed stage or WSE hydrograph at Hardin matches the observed record to within the error tolerance. The calibration then moves to the next section from Hardin to Pearl. Since the WSE at Hardin has already been computed, it is necessary to compute the WSE at Pearl. This will require a systematic adjustment of the conveyance until the recorded WSE hydrograph at Pearl is closely matched. This procedure is repeated for all river sections between adjacent upstream gages. Since there are 11 gages on the lower Illinois River, ten roughness coefficients will be updated during the model calibration. On the Sangamon River, the roughness coefficient for the section between Oakford and the confluence of the Illinois River will be updated also.

Since the UNET model is based on an implicit solution procedure, the boundary condition at the upstream end can be either WSE or flow. Because the USCOE gage at Peoria has only stage records, WSE was prescribed at the upstream end of the study reach. It was also used as the downstream boundary condition at Grafton. The April 1979 and March 1985 floods were selected for calibration and the data for four additional events in 1973, 1974, 1982, and 1993 and were used to verify the calibrated parameters. The results of these simulations are discussed in the following sections.

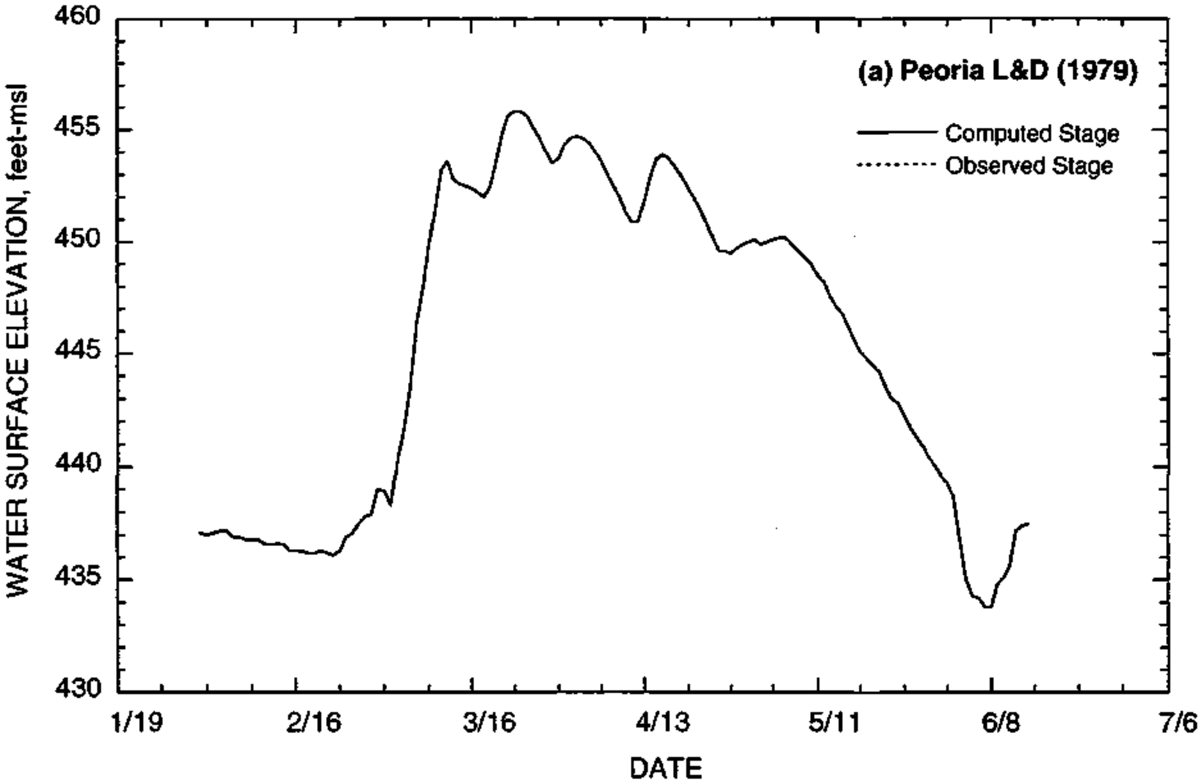
Calibration of 1979 and 1985 Floods

One of the flood events selected for calibration is the May 1979 flood ranked in Table 6 as the fourth highest flow at Meredosia and the sixth highest flow at Kingston Mines. The second event selected is the March 1985 flood ranked in Table 6 as second and third at Meredosia and Kingston Mines, respectively. The flood of May 1943, the highest flood at Meredosia and the second highest at Kingston Mines, was not selected because of missing data in the records of some of the tributaries.

Figure 9 shows the April 1979 WSE hydrographs that were used as upstream boundarycondition at Peoria L&D and downstream boundary condition at Grafton. Figure 10 shows the computed WSE hydrographs obtained from the model calibration for the nine gages between Peoria and Grafton. The computed hydrographs of surface water elevation at Kingston Mines, Havana, and Beardstown [Figures 10(a) - 10(c)] fit the observed records very closely. The differences between the computed and observed WSE are generally less than 0.5 feet around the crest of the hydrographs, the only exception being at Havana. The errors in the computed WSE are larger and generally above 1 foot from La Grange to Grafton. The model underpredicted the WSE at these latter stations, especially on the rising limb of the hydrograph. The computed hydrographs (Figure 10) fit

Table 6. Peak Floods in the Illinois River and Major Tributaries between Grafton and Peoria Lock and Dam

Item	*Macoupin*	*Ill. River Meredosia*	*LaMoine*	*Sangamon*	*Spoon*	*Ill. River Kingston Mines*	*Mackinaw*
Rank	1	1	18	1	28	2	11
Q_p	40,000	123,000	14,500	123,000	12,900	83,100	18,200
Date	5/18/43	5/26/43	5/21/43	5/20/43	5/20/43	5/23/43	5/19/43
Rank	10	2	1	21	3	3	10
Q_P	19,400	122,000	28,800	26,500	29,200	78,800	18,400
Date	2/24/85	3/10/85	3/7/85	2/25/85	3/6/85	3/6/85	2/25/85
Rank	3	3	6	2	9		1 1
Q_p	26,700	112,000	21,000	68,700	21,000	88,800	51,000
Date	12/4/82	12/12/82	12/5/82	12/5/82	4/4/83	12/7/82	12/6/82
Rank	2	4	34	3	30	6	6
Q_p	27,800	111,000	8,120	55,900	12,600	72,300	23,600
Date	4/12/79	4/19/79	4/12/79	4/15/79	3/31/79	3/24/79	3/5/79
Rank	22	5	29	6	1	7	30
Q_P	12,800	110,000	10,000	42,900	36,400	71,900	7,910
Date	1/21/74	6/29/74	6/2/74	6/25/74	6/24/74	5/25/74	6/5/74
Rank	31	6	28	29	17	4	8
Q_P	9,140	104,000	10,700	23,700	16,900	77,200	20,000
Date	2/21/82	3/24/82	3/16/82	3/20/82	7/21/82	3/22/82	3/12/82
Rank	20	7	22	4	21	14	4
Q_P	13,300	102,000	12,400	45,800	16,000	63,300	29,700
Date	4/23/73	5/2/73	4/22/73	4/25/73	4/24/73	4/25/73	4/21/73



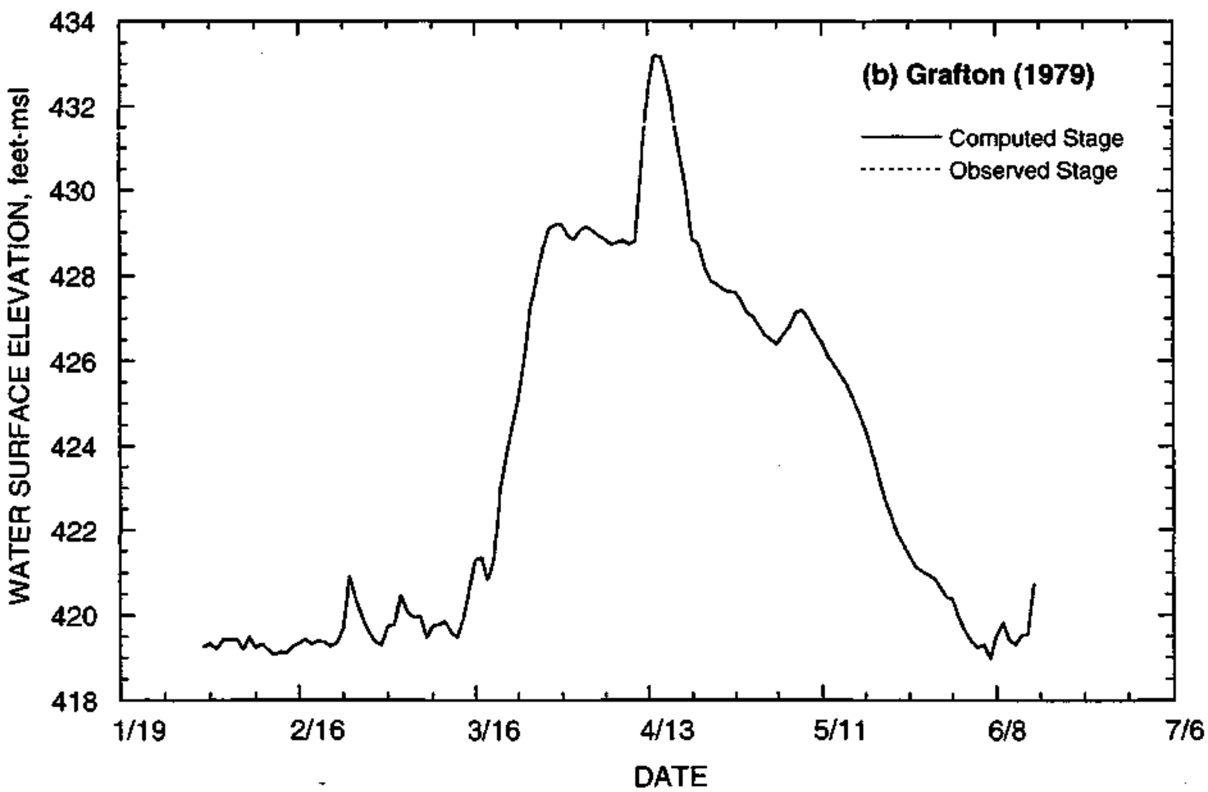


Figure 9. Water surface elevation hydrographs used as (a) upstream boundary condition at Peoria Lock and Dam and (b) downstream boundary condition at Grafton

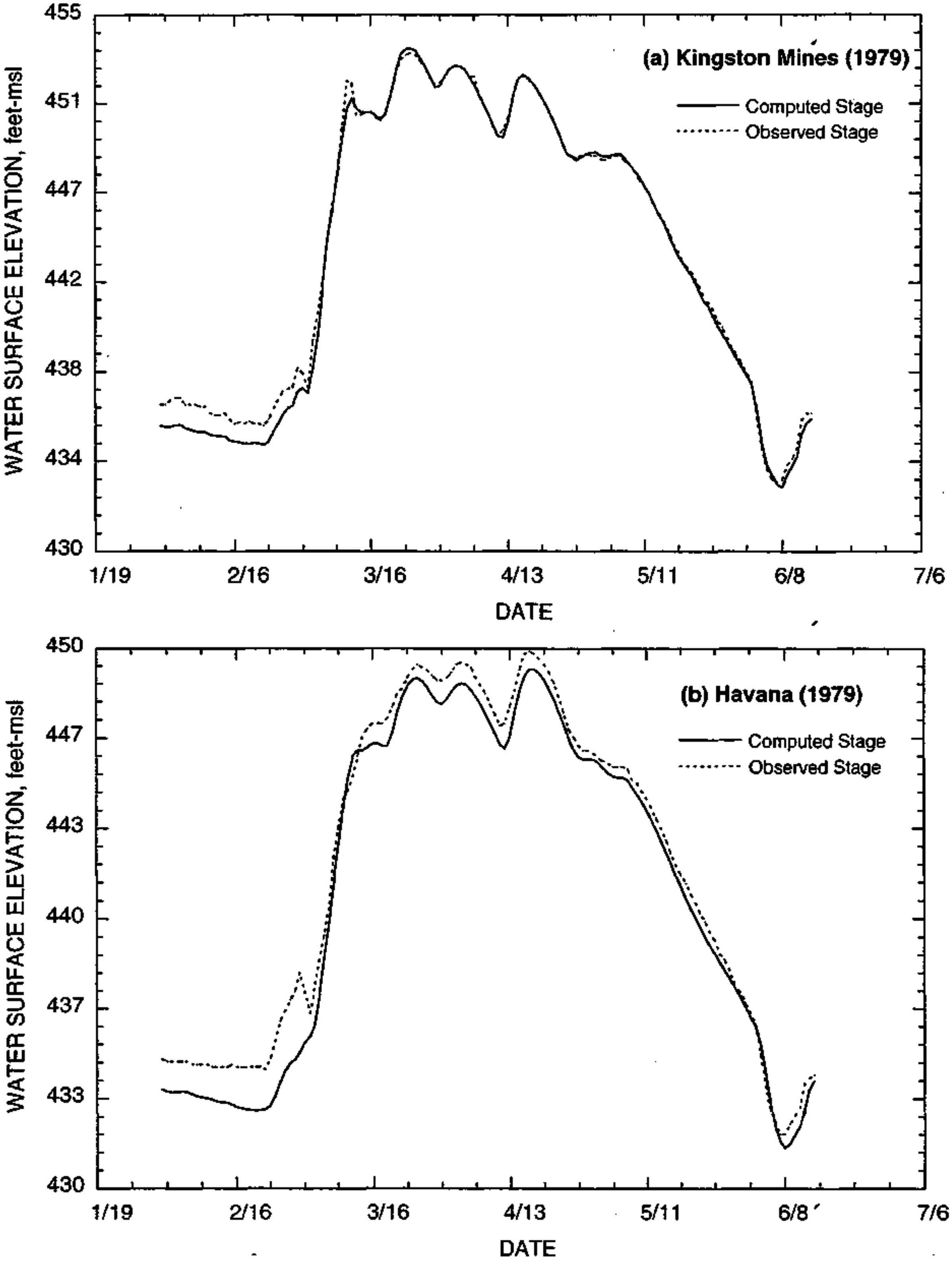


Figure 10. Computed and observed water surface elevation hydrographs at (a) Kingston Mines, (b) Havana, (c) Beardstown, (d) La Grange Lock and Dam, (3) Meredosia, (f) Valley City, (g) Florence, (h) Pearl, and (I) Hardin

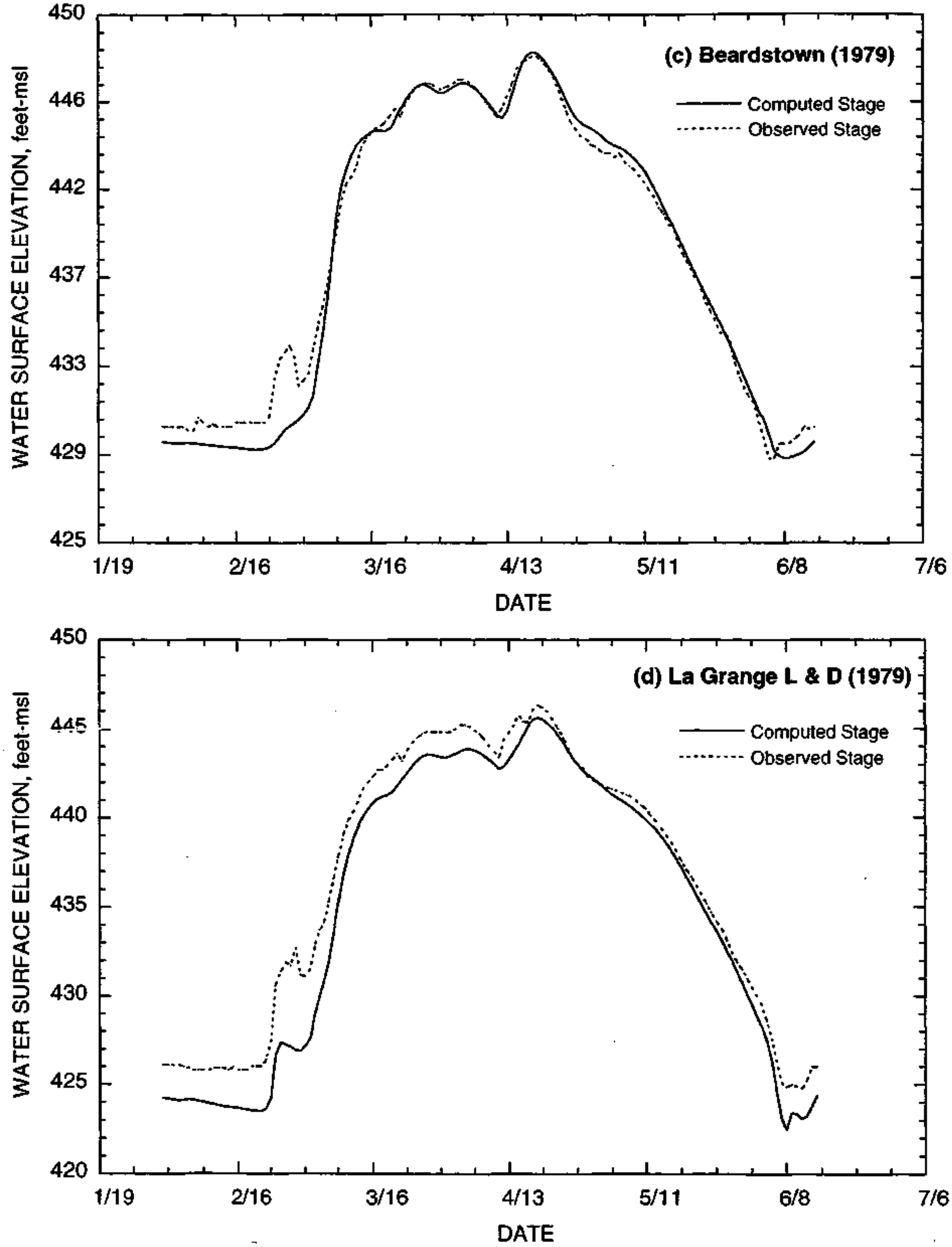
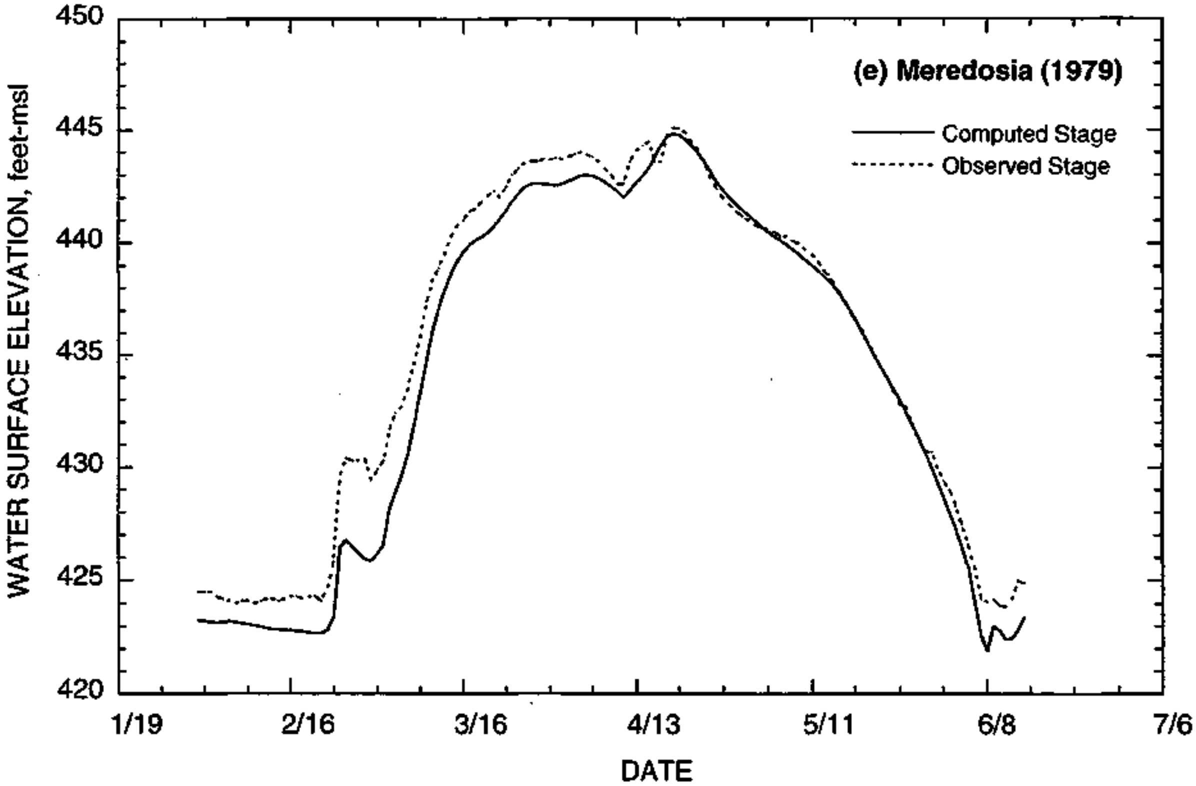


Figure 10 (continued)



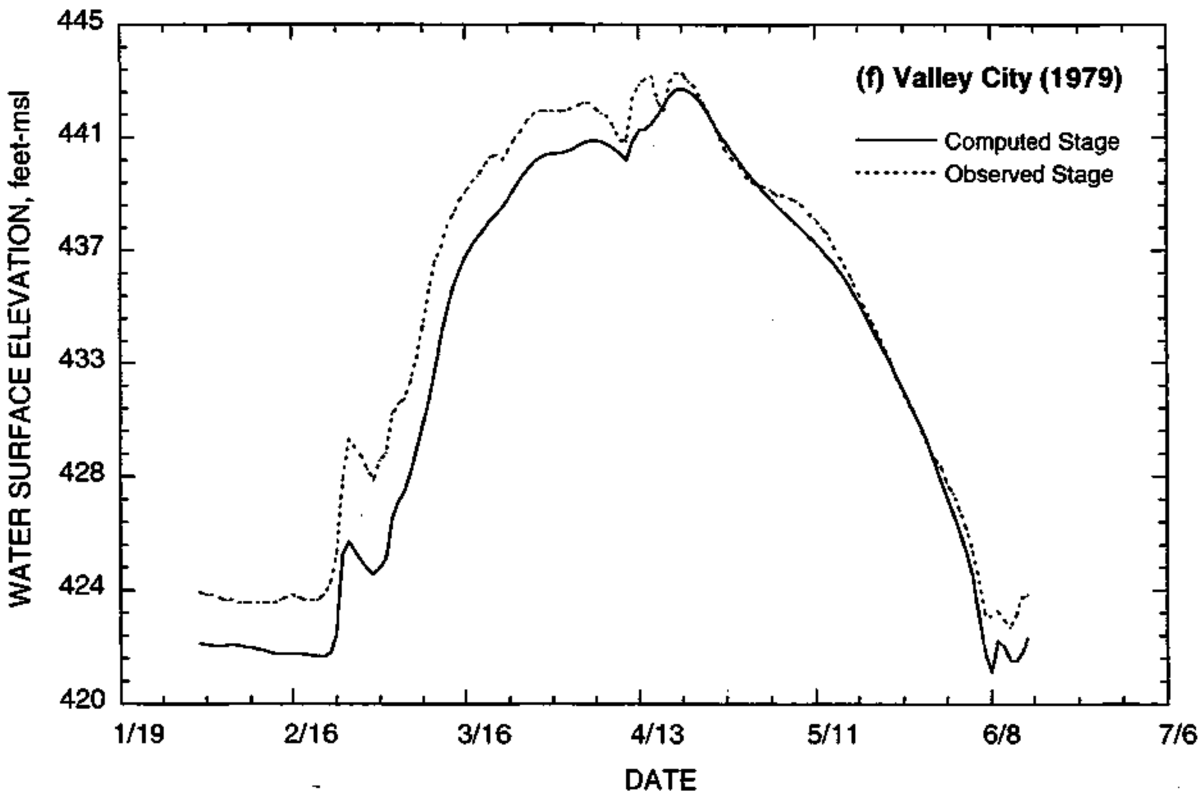
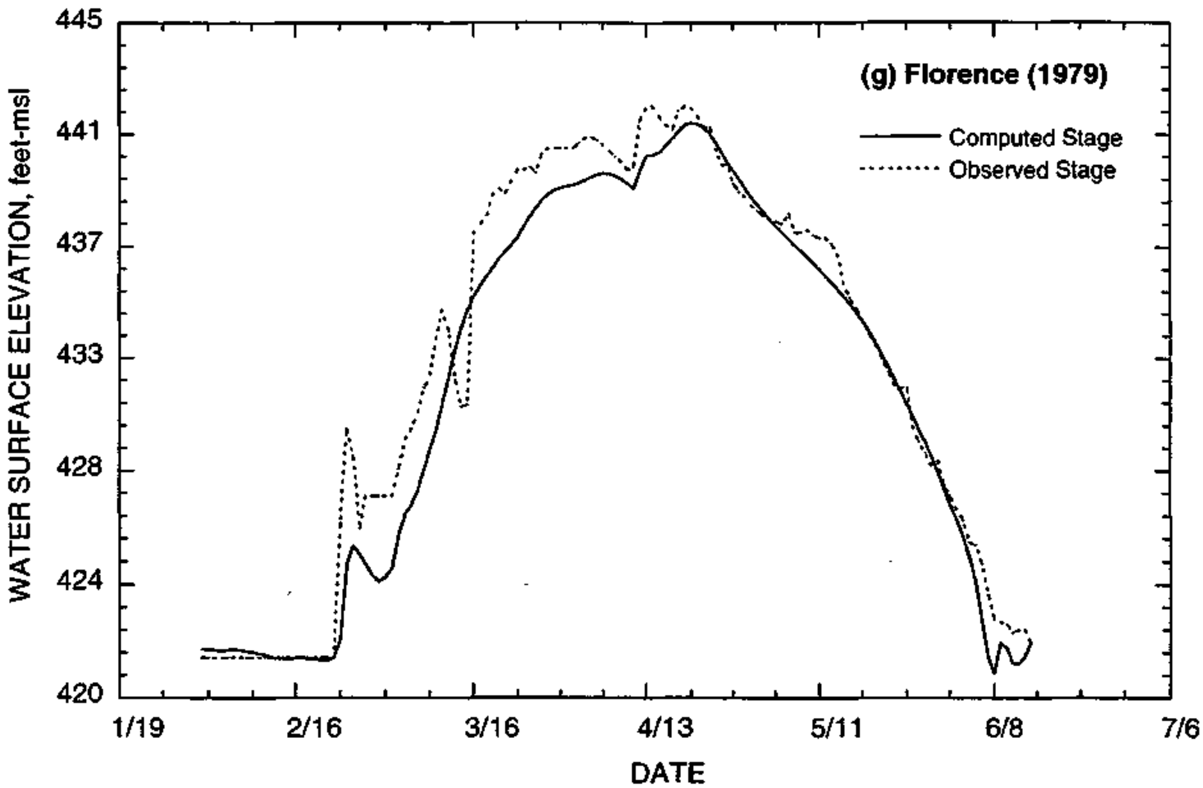


Figure 10. (continued).



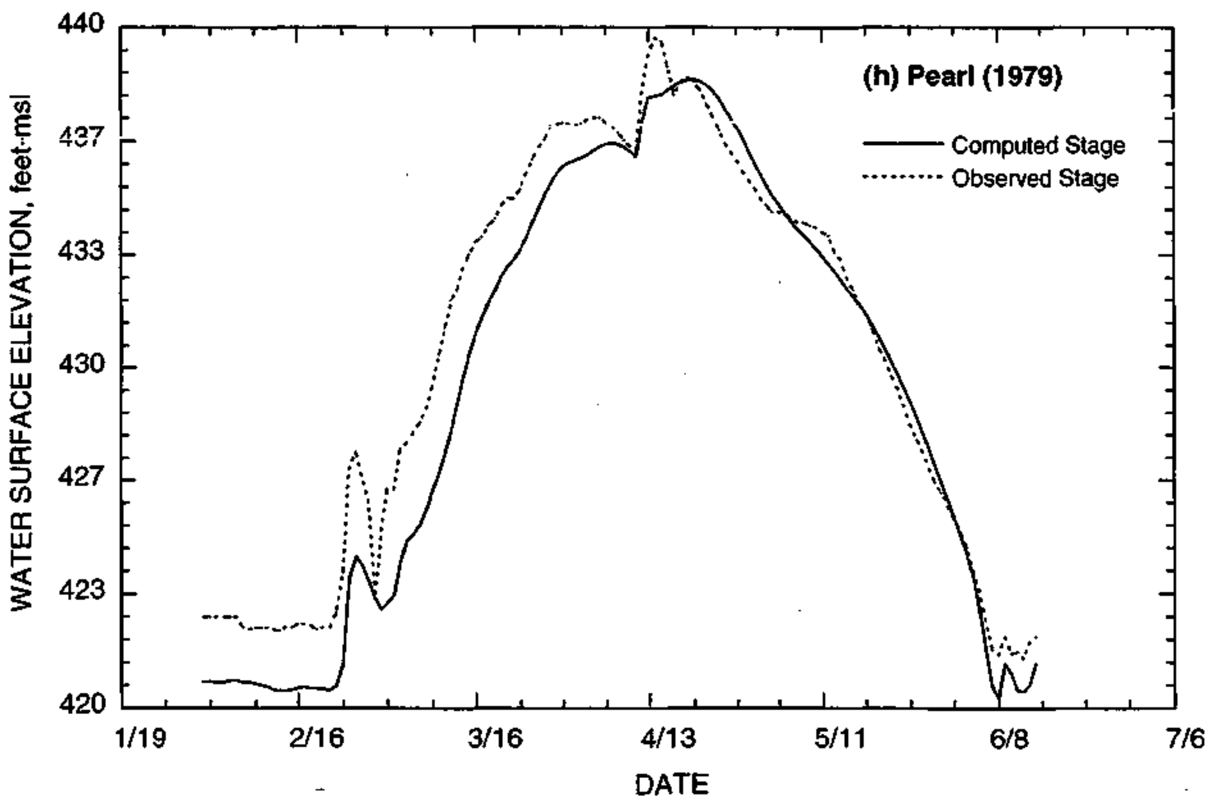


Figure 10. (continued)

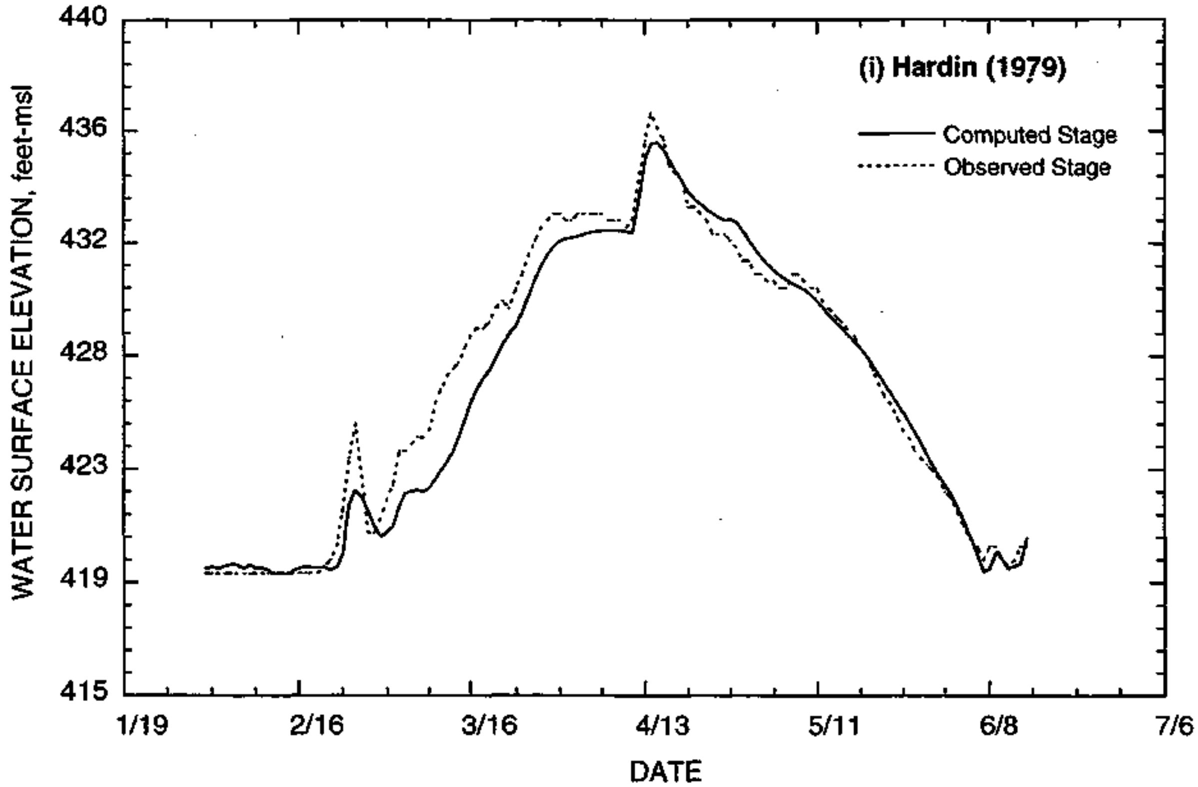


Figure 10. (concluded)

the observed WSE much better on the falling limb. Peak WSEs are also underpredicted, but the underestimations are within 1 foot at these downstream gages except Pearl.

Since the model predictions degenerate from La Grange L&D downstream, it seems that the UNET model cannot effectively simulate the operations of the lock and dam and the flow through the structure. This apparent problem could be avoided by setting up the study reach as two separate subreaches: the La Grange Pool and the Alton Pool. The subdivision of the lower Illinois River into two reaches implies that La Grange L&D will serve as the downstream boundary for the La Grange Pool as well as the upstream boundary for the Alton Pool. Figure 11 shows the April 1979 WSE hydrograph that was used as the boundary condition at La Grange, and Figure 12 shows the plots of the station hydrographs resulting from the two-reach model calibrations. Table 7 shows the coefficient of determination values, and Figure 13 shows the computed peak WSE for the two-reach model and the computed results for the one-reach simulations and the observed peak WSEs at the 11 stations for comparison. It should be noted that the peak WSE profile is not a snapshot of the water surface elevation at a particular instance in time. It is, however, a plot of the peaks of the WSE hydrographs at each of the cross sections. The WSE profile for the two-reach model (Figure 13) more closely fit the observed elevations than the one-reach profile. The two computed profiles are relatively indistinguishable from Peoria to Havana but differ downstream from Havana, the greatest difference occurring between Beardstown and Meredosia. The maximum difference in either of the computed profiles and the observed peak WSE occurred around Meredosia.

Because of the improvement in the computation results for the two-reach model, the model calibration for the March 1985 flood was also carried out for both one-reach and two-reach models. The coefficient of determination for the stations is shown in the second column of Table 7, the computed WSE profiles are shown in Figure 14 and the station hydrographs are shown in Figure 15. Figure 14 also shows the recorded peak WSE at the gages. Both of the computed profiles in the figure seemed to fit the observed data very well. The one-reach peak WSE profile provides a better fit to the observed data while the two-reach simulations slightly over-predicted the peak WSE at the stations. The differences between the two simulated profiles, and the differences between either profile

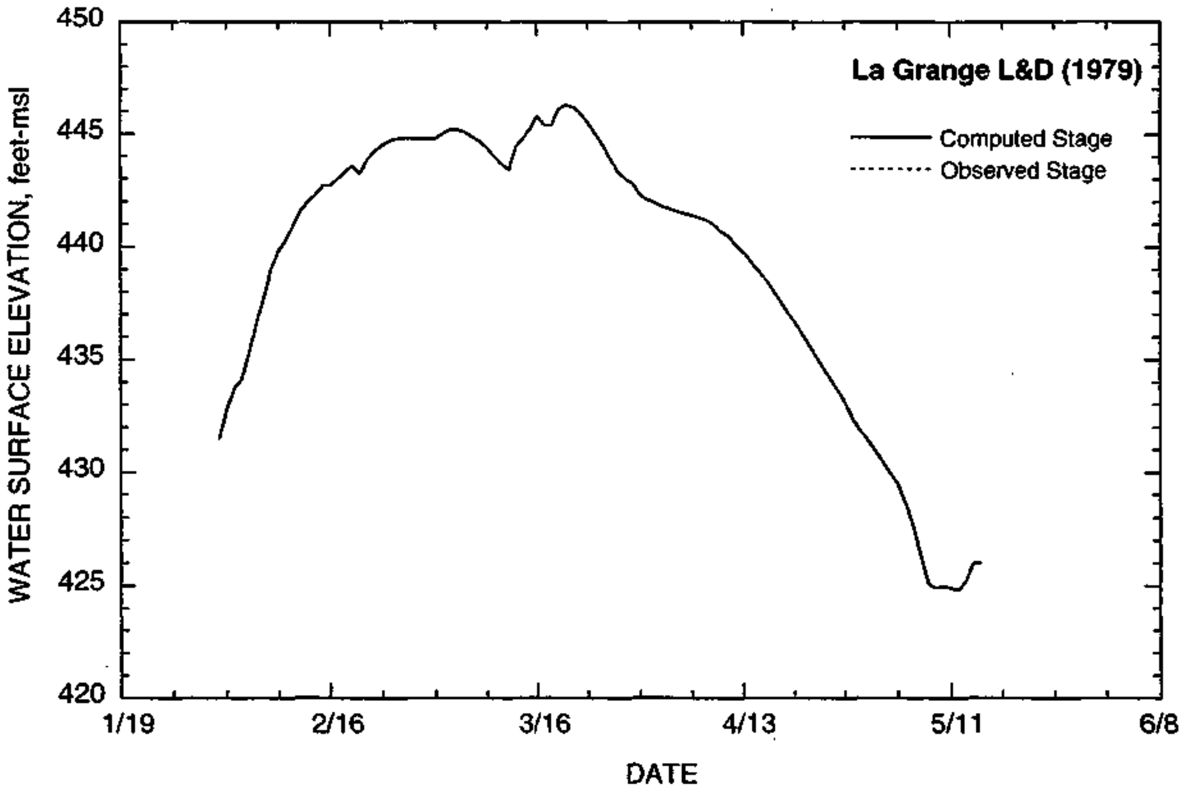


Figure 11. Water surface elevation hydrograph used as boundary condition at La Grange Lock and Dam for the two-reach model

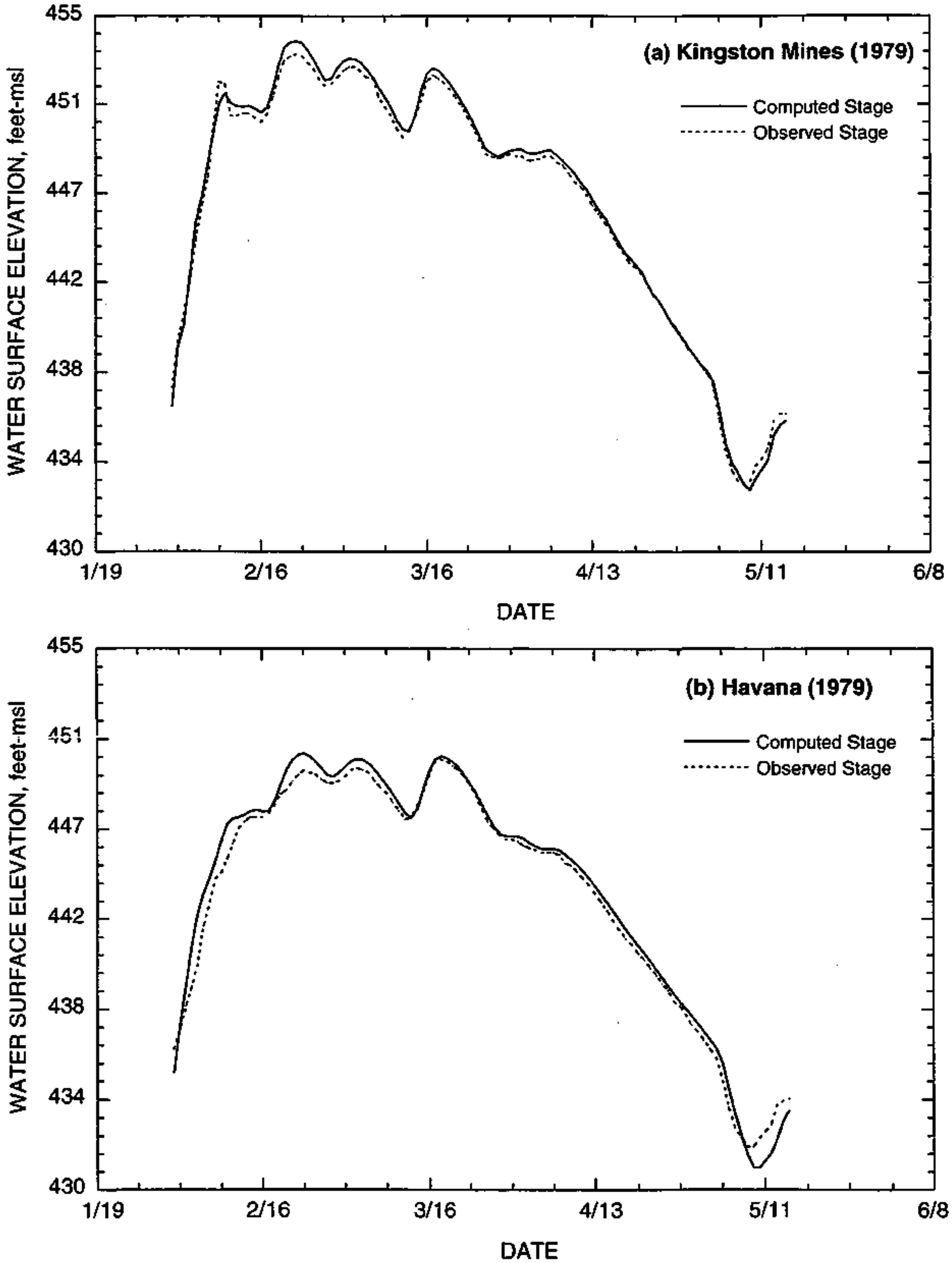
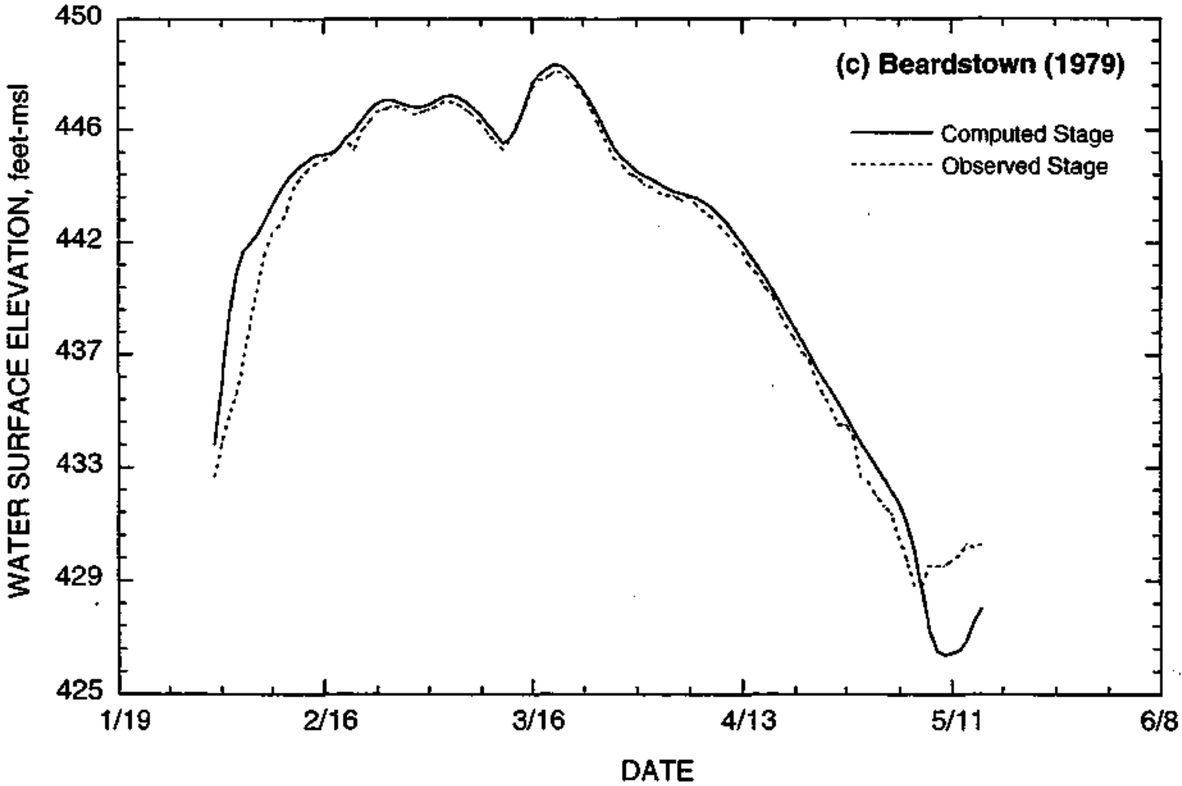


Figure 12. Water surface elevation hydrographs computed (two-reach model) and observed at (a) Kingston Mines, (b) Havana, (c) Beardstown, (d) Meredosia, (e) Valley City, (f) Florence, (g) Pearl, and (h) Hardin.



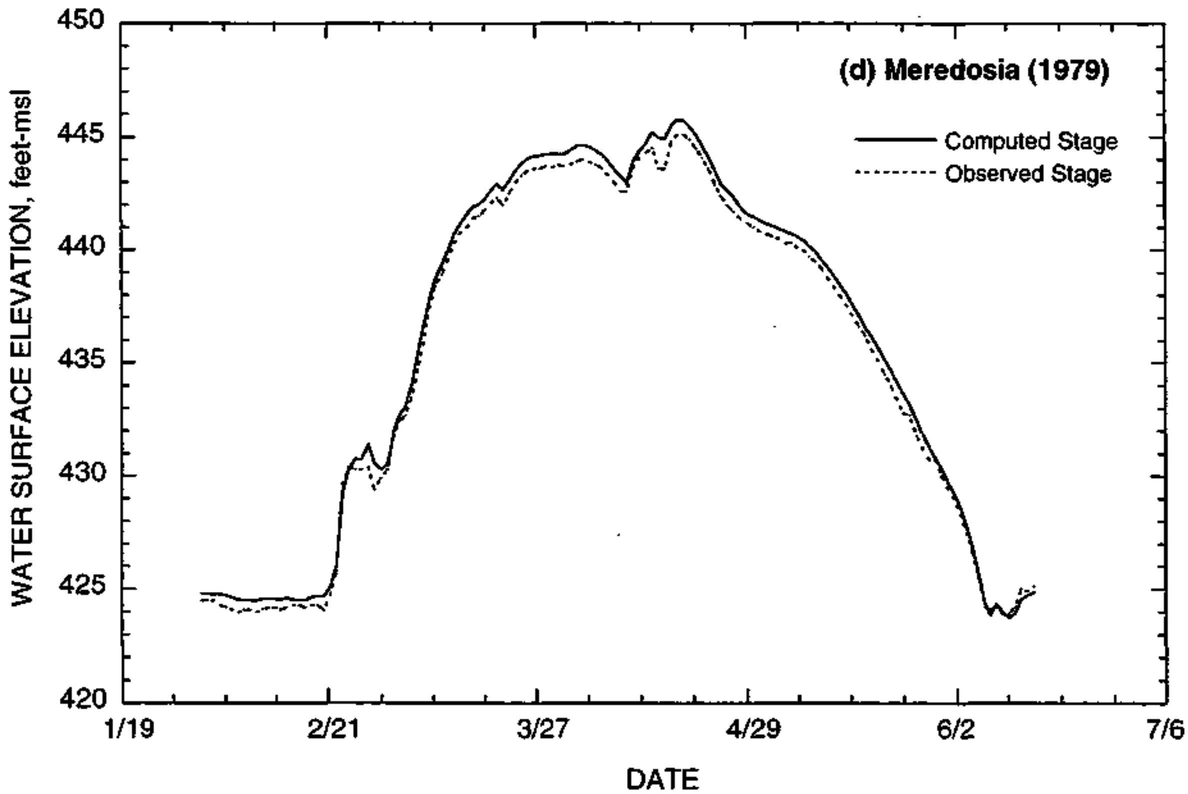
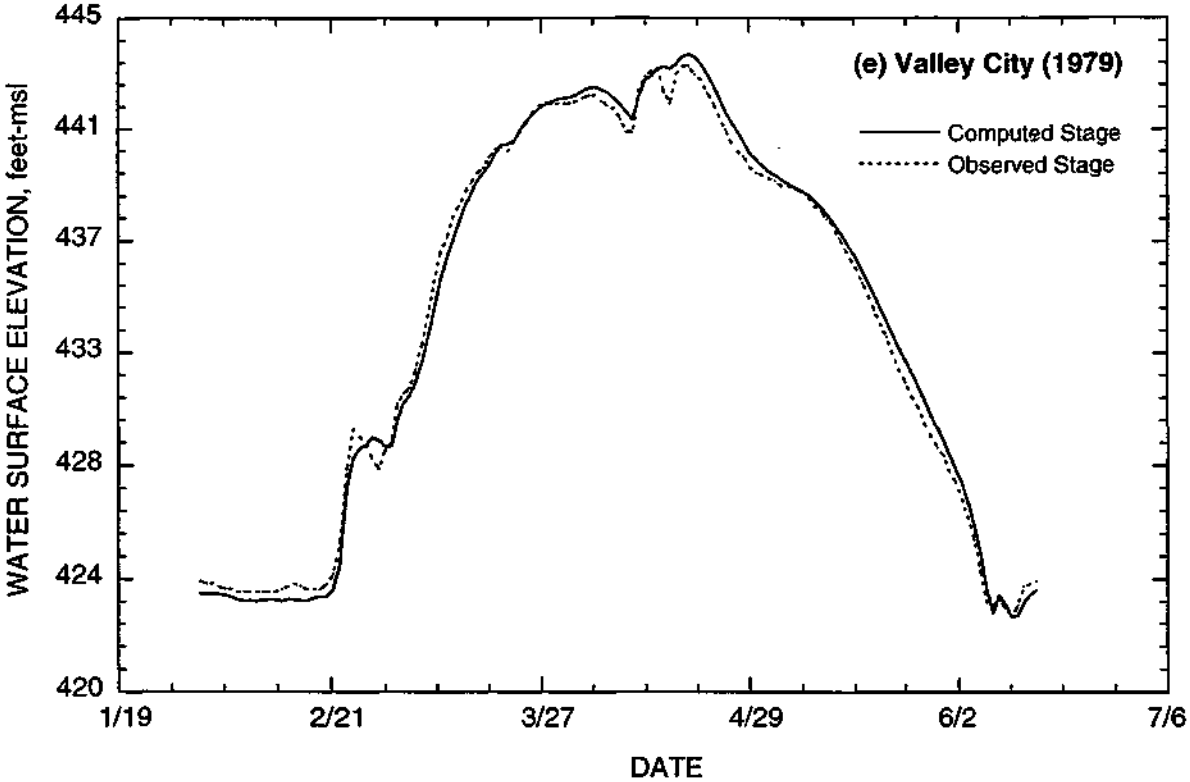


Figure 12 (continued)



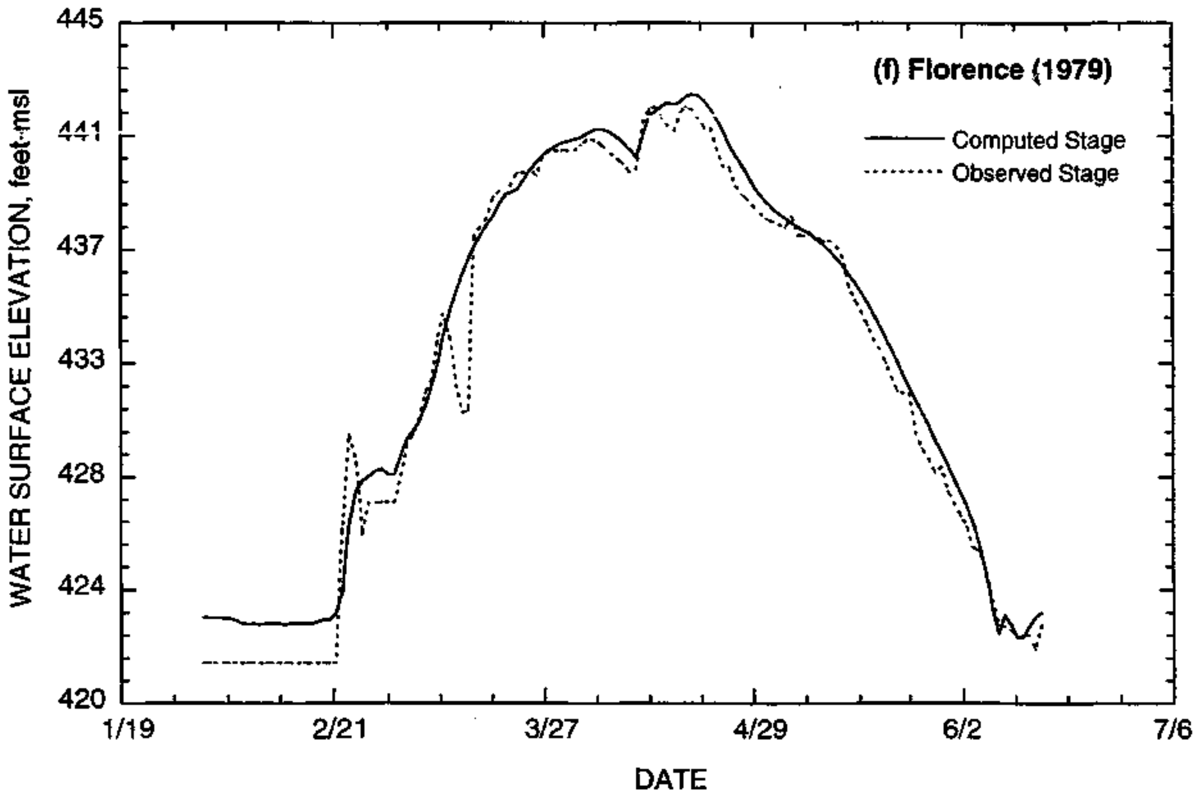


Figure 12. (continued)

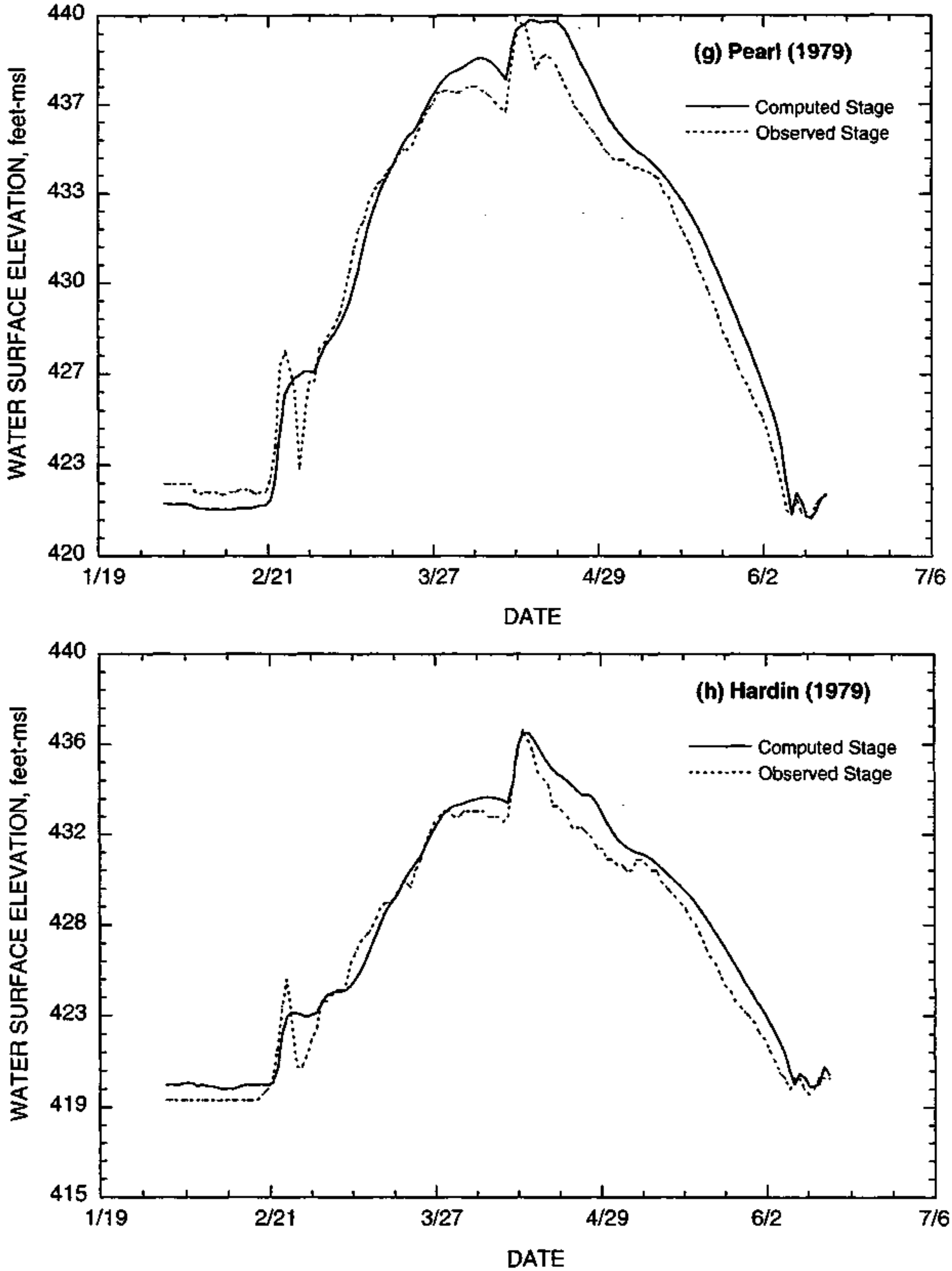


Figure 12. (concluded)

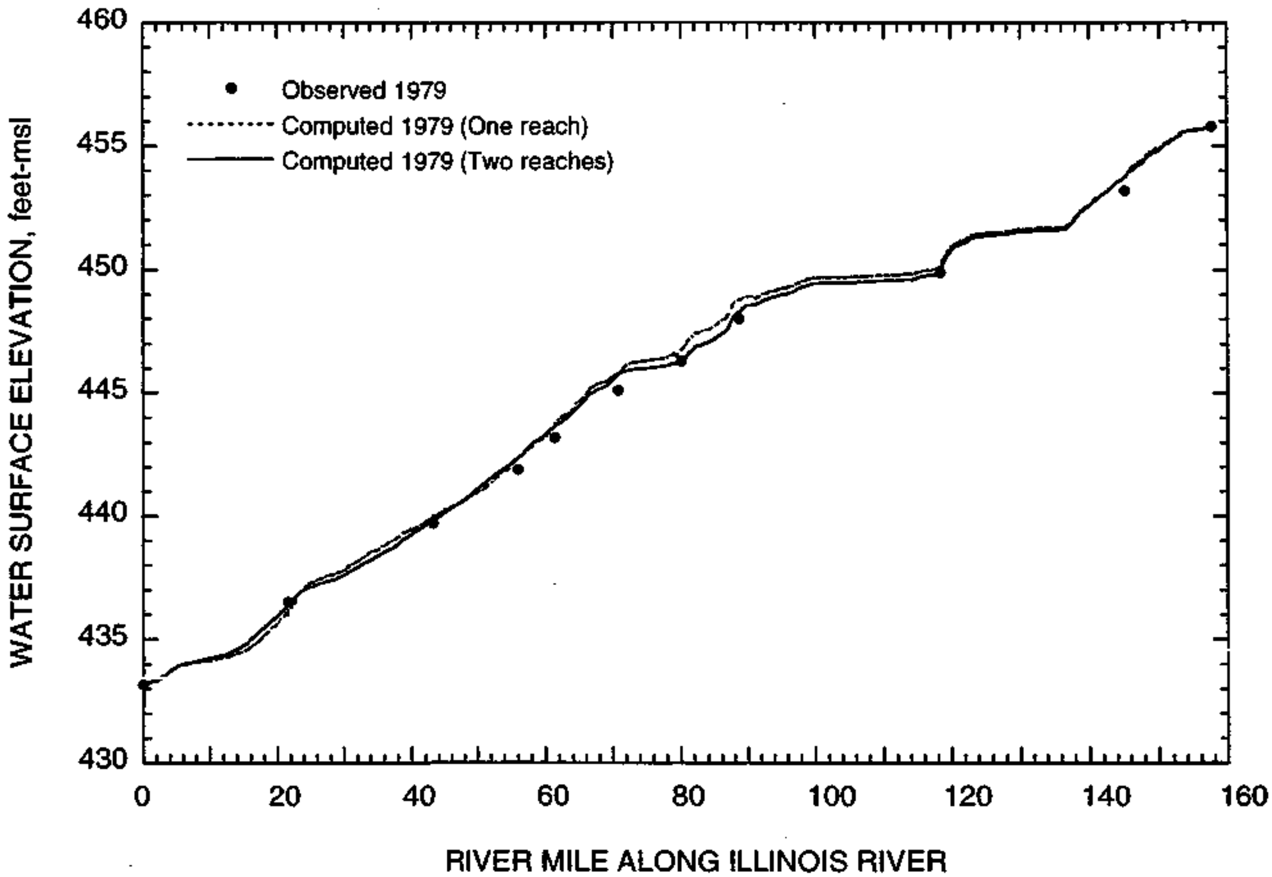


Figure 13. Comparison of peak water surface elevation profiles for one-reach and two-reach models with observed peak stages for the 1979 flood at gaging stations along the lower Elinois River.

Table 7. Coefficient of Determination Values for the Observed and Computed Stage Hydrographs Using Two-Reach Model

	Calibration		*Verification*		
Station	*1979*	*1985*	*1982*	*1974*	*1973*
Peoria L&D TW					
Kingston Mines	0.9979	0.9941	0.9978	0.9990	0.9975
Havana	0.9904	0.9820	0.9970	0.9899	0.9928
Beardstown	0.9618	0.9662	0.9888	0.9469	0.9967
La Grange L&D TW					
Meredosia	0.9991	0.9990	0.9963	0.9987	0.9973
Valley City	0.9960	0.9921	0.9768	0.9871	0.9833
Florence	0.9821	0.9872	0.9614	0.9792	0.9669
Pearl	0.9808	0.9759	0.9069	0.9560	0.9572
Hardin	0.9785	0.9612	0.8813	0.9204	0.9660
Grafton					
Number of Days	135	151	45	107	151

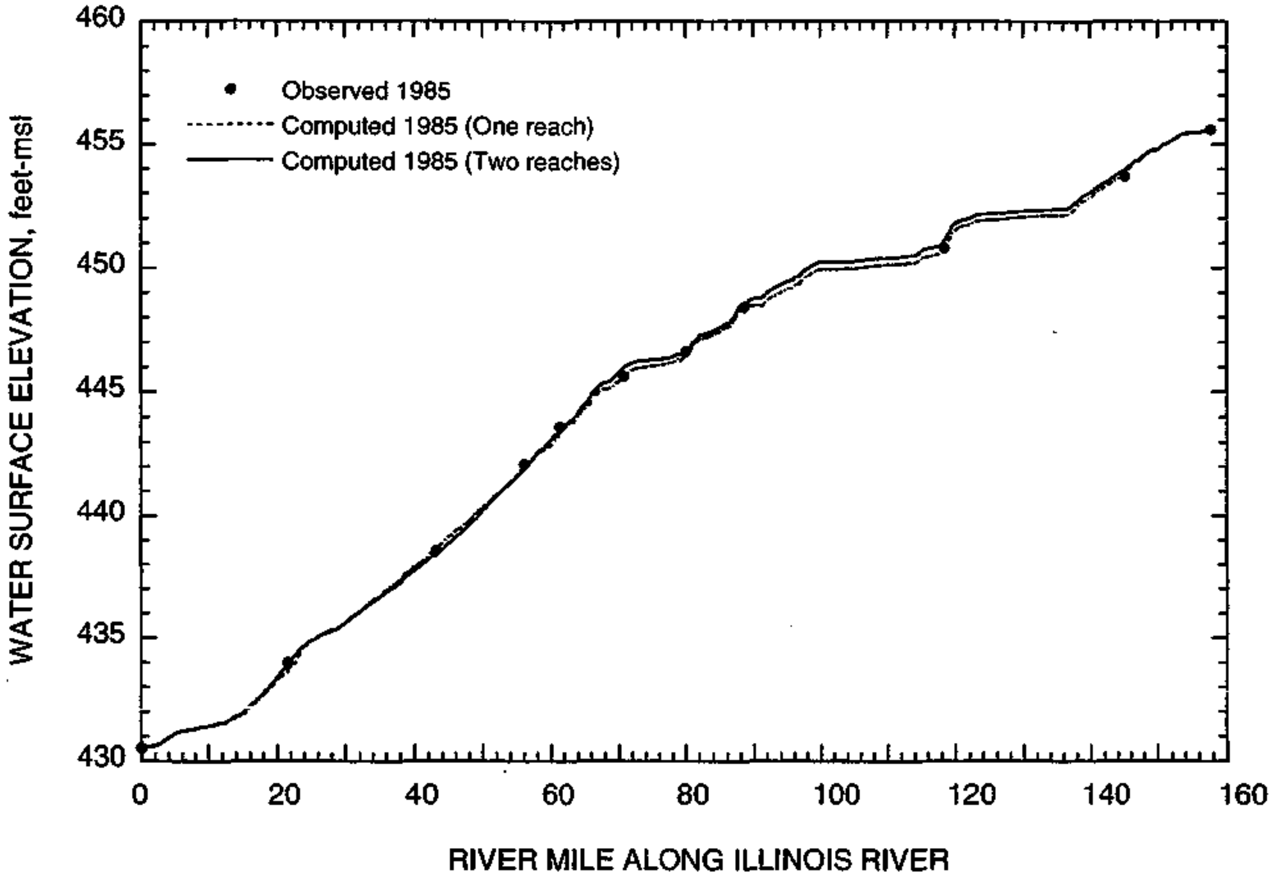


Figure 14. Comparison of peak water surface elevation profiles for one-reach and two-reach models with observed peak stages for the 1985 flood at gaging stations along the lower Illinois River

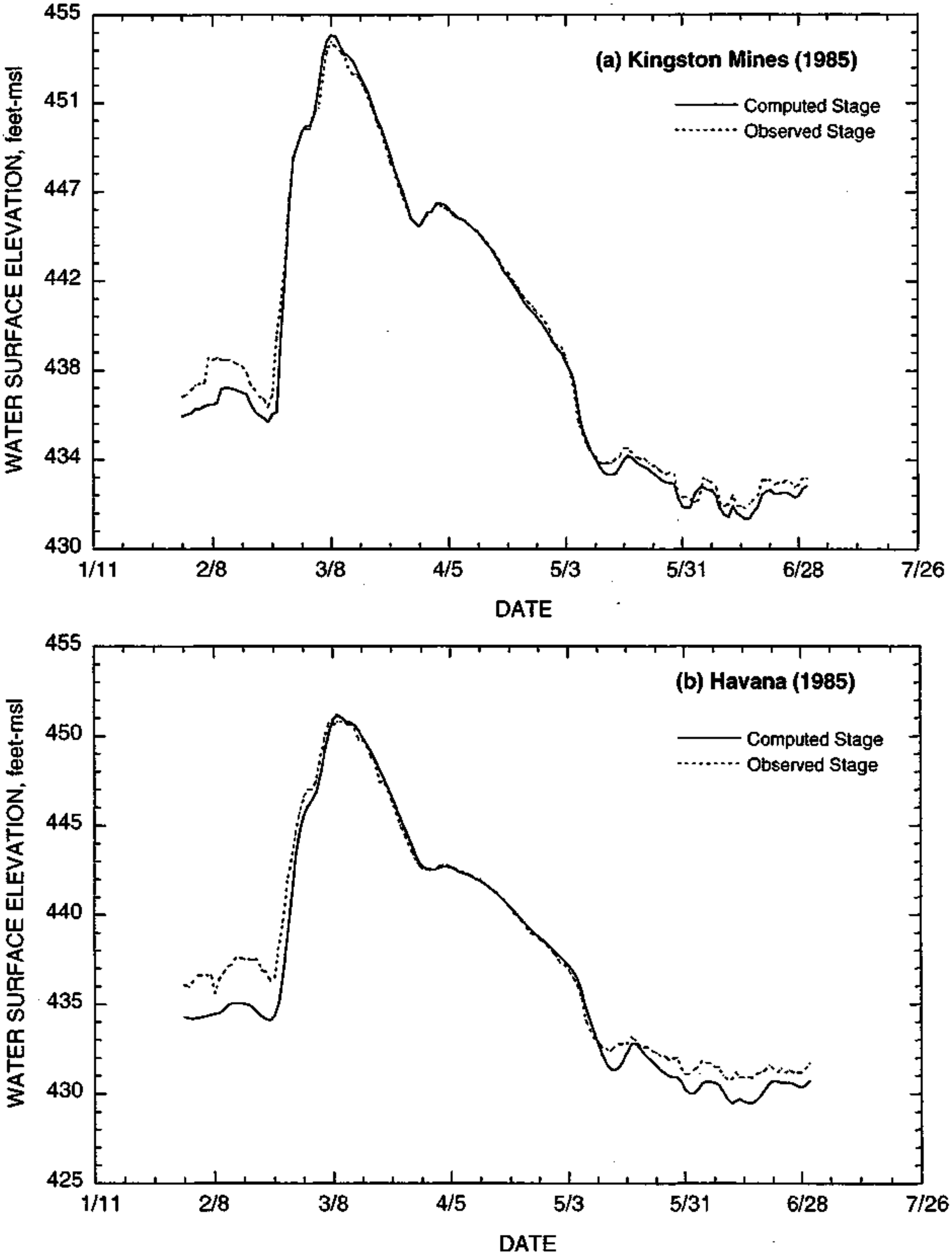
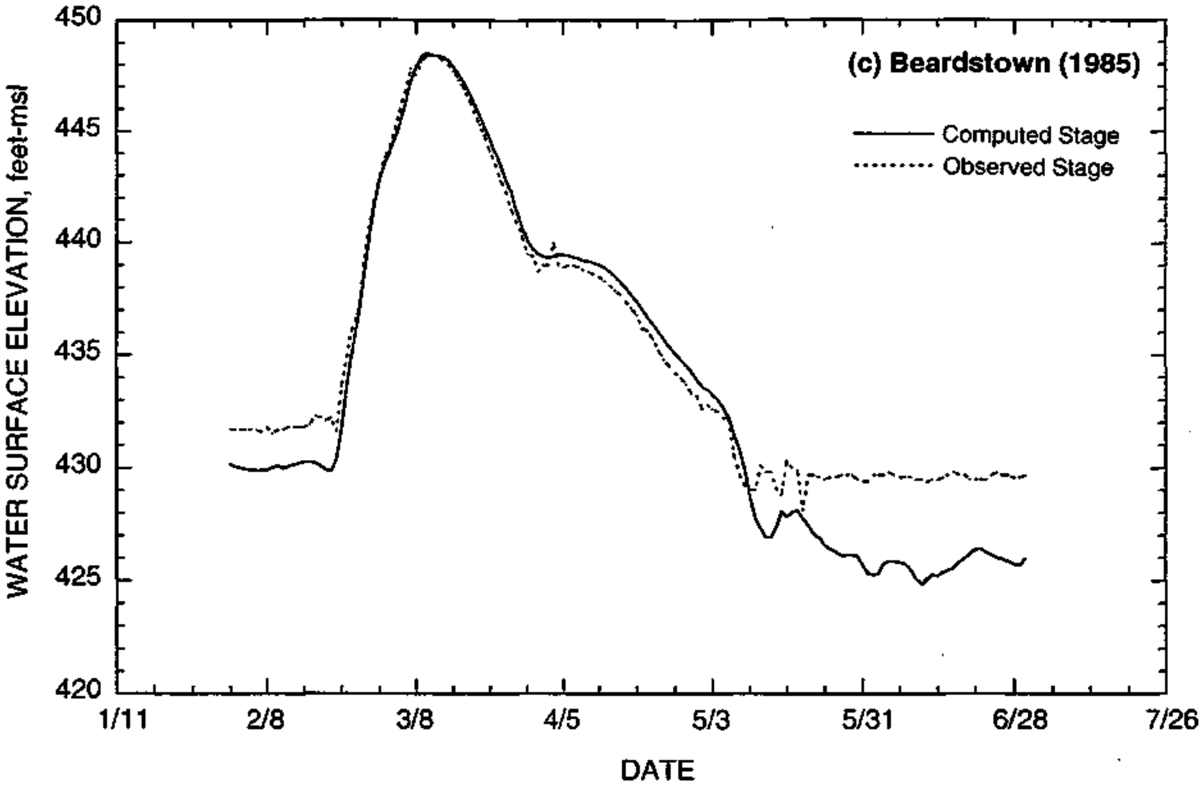


Figure 15. Water surface elevation hydrographs computed (two-reach model) and observed at (a) Kingston Mines, (b) Havana, (c) Beardstown, (d) Meredosia, (e) Valley City, (f) Florence, (g) Pearl, and (h) Hardin.



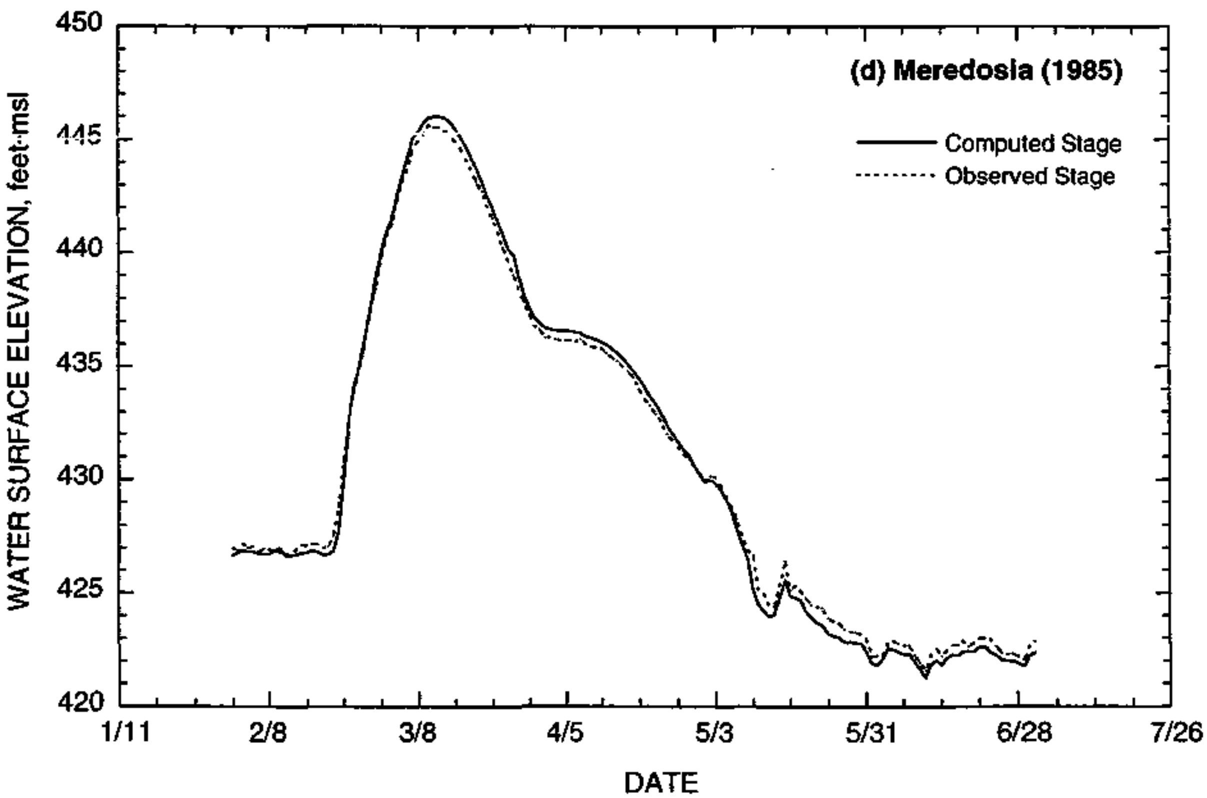


Figure 15. (continued)

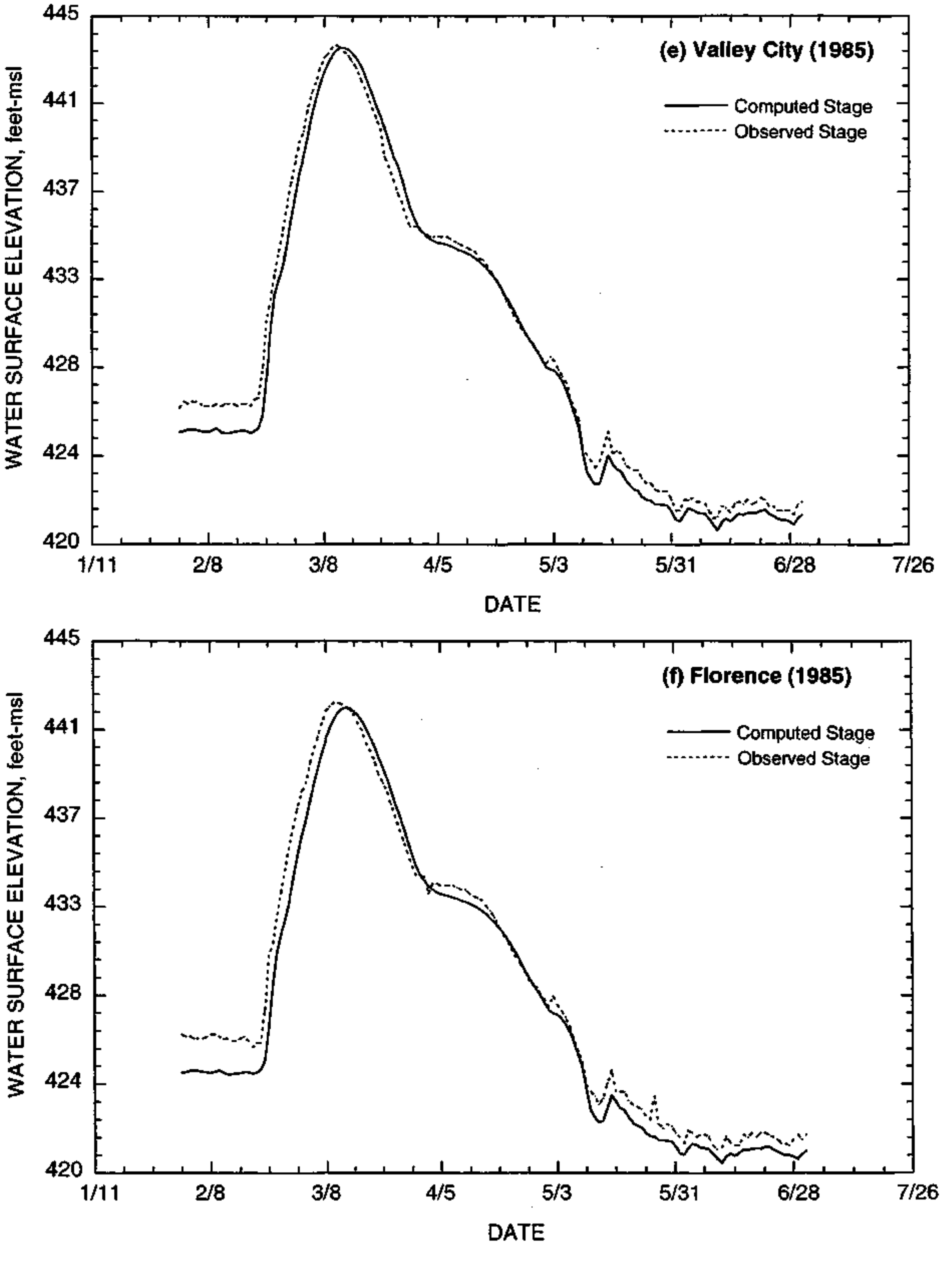
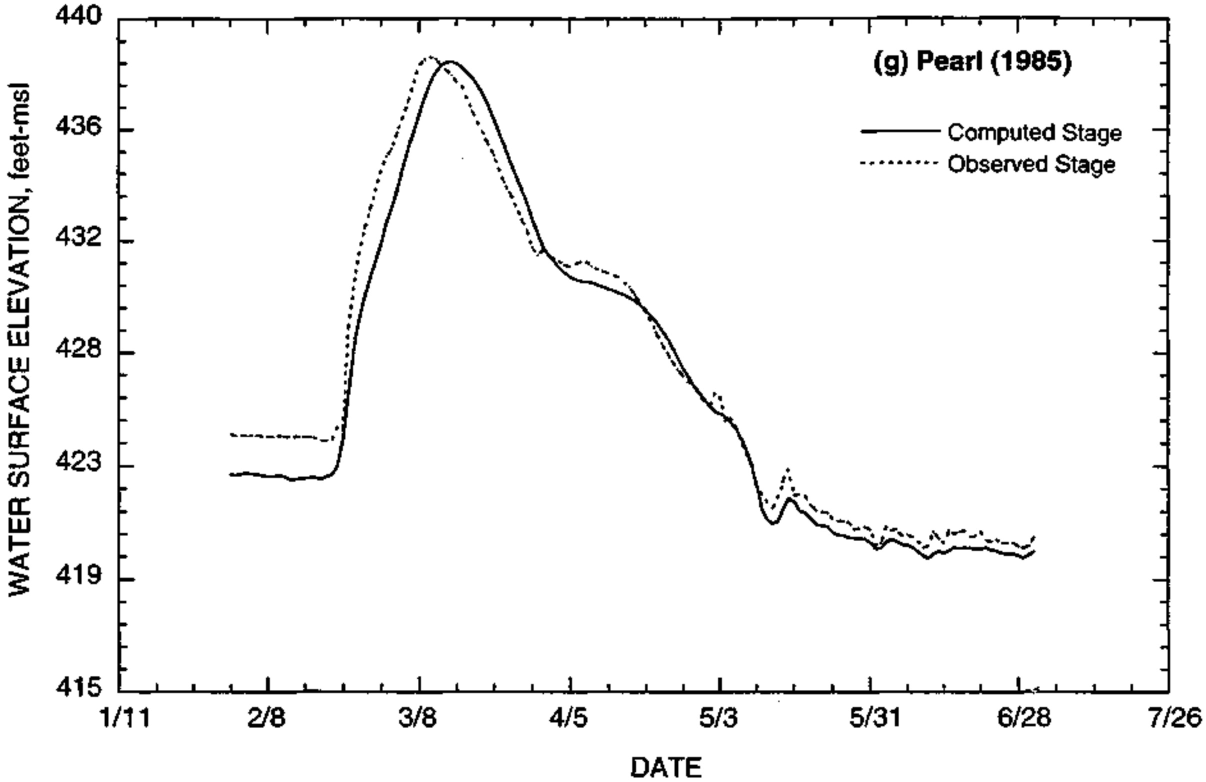


Figure 15. (continued)



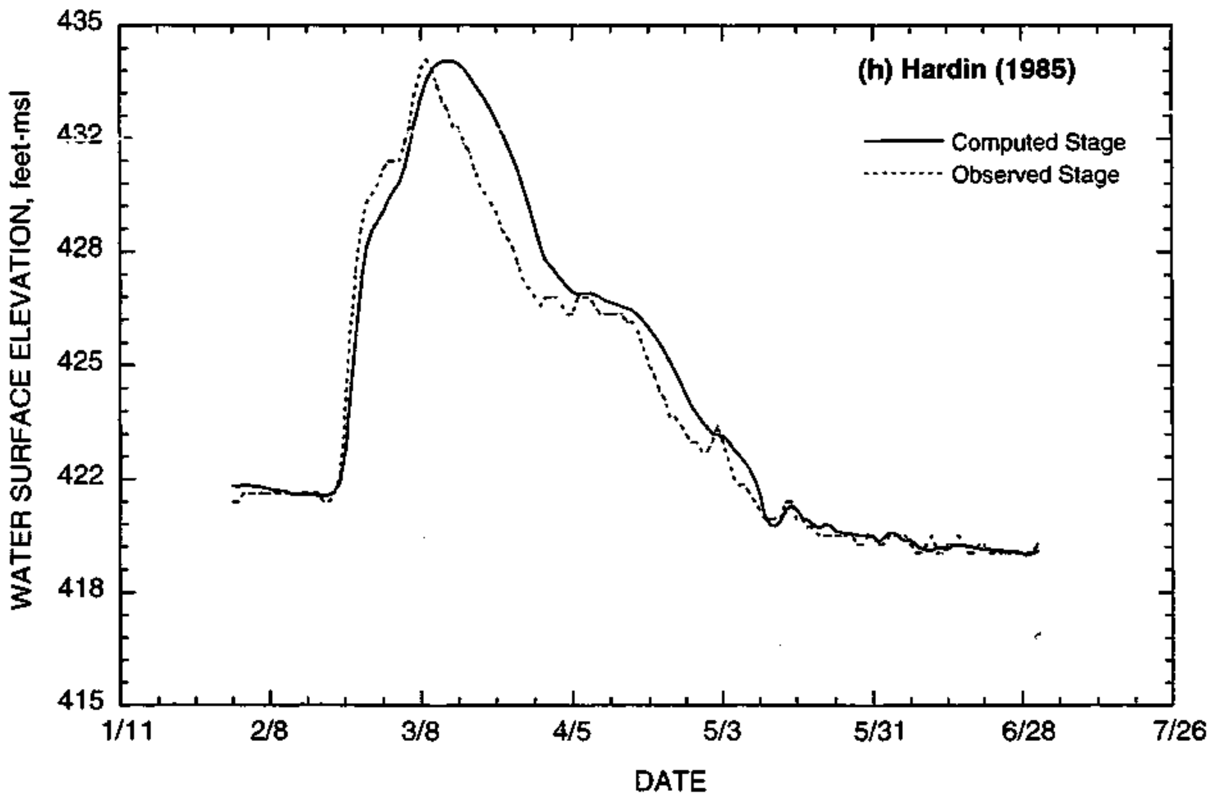


Figure 15. (concluded)

and the observed peak WSE are less than 0.5 feet. A comparison of the 1979 and 1985 coefficient of determination values in Table 7 indicates that the two-reach model calibrations for the 1979 flood generally provided better representation of observed WSE hydrographs than the 1985 simulations. However, by examining the computed peak WSE profile for 1985 (see Figure 14), the two-reach calibration for the 1985 event is shown to be better than the 1979 calibration (Figure 13). As a result of these observations, the Manning roughness coefficients, obtained from the two-reach calibration for both floods events, were averaged to obtain the roughness coefficient values that will be used in all subsequent simulations. Table 8 lists these roughness coefficient values. The roughness coefficient is 0.02 for the channel areas of all cross sections. The value of the roughness coefficients for the overbank areas varies between 0.03 and 0.1. A value of 1.0 has been used for the areas behind the levees to prevent any flow in this region.

Model Verification with Other Floods

Using the roughness coefficient values obtained from the calibration of the 1979 and 1985 floods, the model was applied to simulate the December 1982, June 1974, April 1973, and July 1993 flood events. These events are ranked as third, fifth, seventh, and twelfth at Meredosia and as first, seventh, fourteenth, and eighteenth at Kingston Mines, respectively. The computed two-reach peak WSE profiles and the corresponding WSE hydrographs at the gages are plotted in Figures 16-21. Figures 16, 18, and 20 are profiles for the 1982, 1974, and 1973 events, respectively, and Figures 17, 19, and 21 show the corresponding WSE hydrographs at the gage locations. Figure 22 shows the peak WSE profile for the 1993 flood.

The computed two-reach peak WSE profiles for the 1982 flood, shown in Figure 16, fit the observed peaks better than the one-reach results. The computed profiles are similar except in the Alton Pool, most especially from Hardin to Grafton,where they differ by more than 1 foot. The WSE hydrographs for the two-reach simulations are plotted in Figure 17. The computed and observed hydrographs at Kingston Mines, Beardstown, and Meredosia matched the observed data very closely. Although the errors in the computed peak WSE at Valley City, Florence, and Pearl are within 0.5 feet, the predicted

Table 8. Manning Roughness Coefficients for Reaches along the Illinois River

River reach	*Upstream-downstream River miles*	*Roughness coefficient Channel*	*Left bank*	*Right bank*
Peoria L&D - Kingston Mines	157.8-144.4	0.02	0.10	0.10
Kingston Mines - Havana	144.4-119.6	0.02	0.10	0.03-0.10
Havana - Beardstown	119.6-88.8	0.02	0.10	0.03-0.10
Beardstown - La Grange L&D	88.8-80.2	0.02	0.03-0.1	0.10-1.00
La Grange L&D - Meredosia	80.2-71.3	0.02	0.03-0.1	0.10-1.00
Meredossia - Valley City	71.3-61.3	0.02	0.03-0.1	0.10-1.00
Valley City - Florence	61.3-56.0	0.02	0.03-0.1	0.10-1.00
Florence - Pearl	56.0-43.2	0.02	0.03-0.1	0.10-1.00
Pearl-Hardin	43.2-21.5	0.02	0.03-0.1	0.10-1.00
Hardin - Grafton	21.5-0.0	0.02	0.03-0.1	0.10-1.00

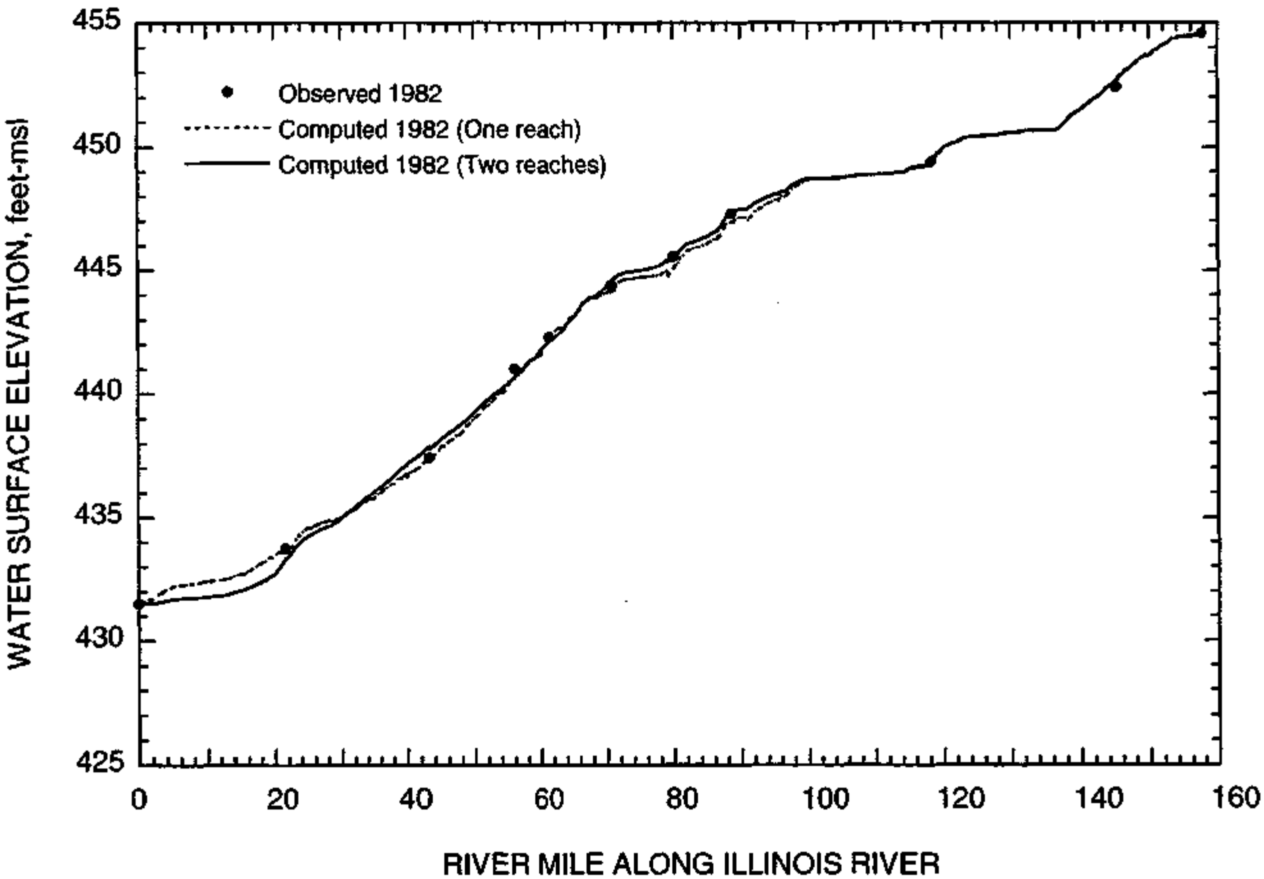


Figure 16. Comparison of peak water surface elevation profiles for one-reach and two-reach models with observed peak stages for the 1982 flood at gaging stations along the lower Illinois River

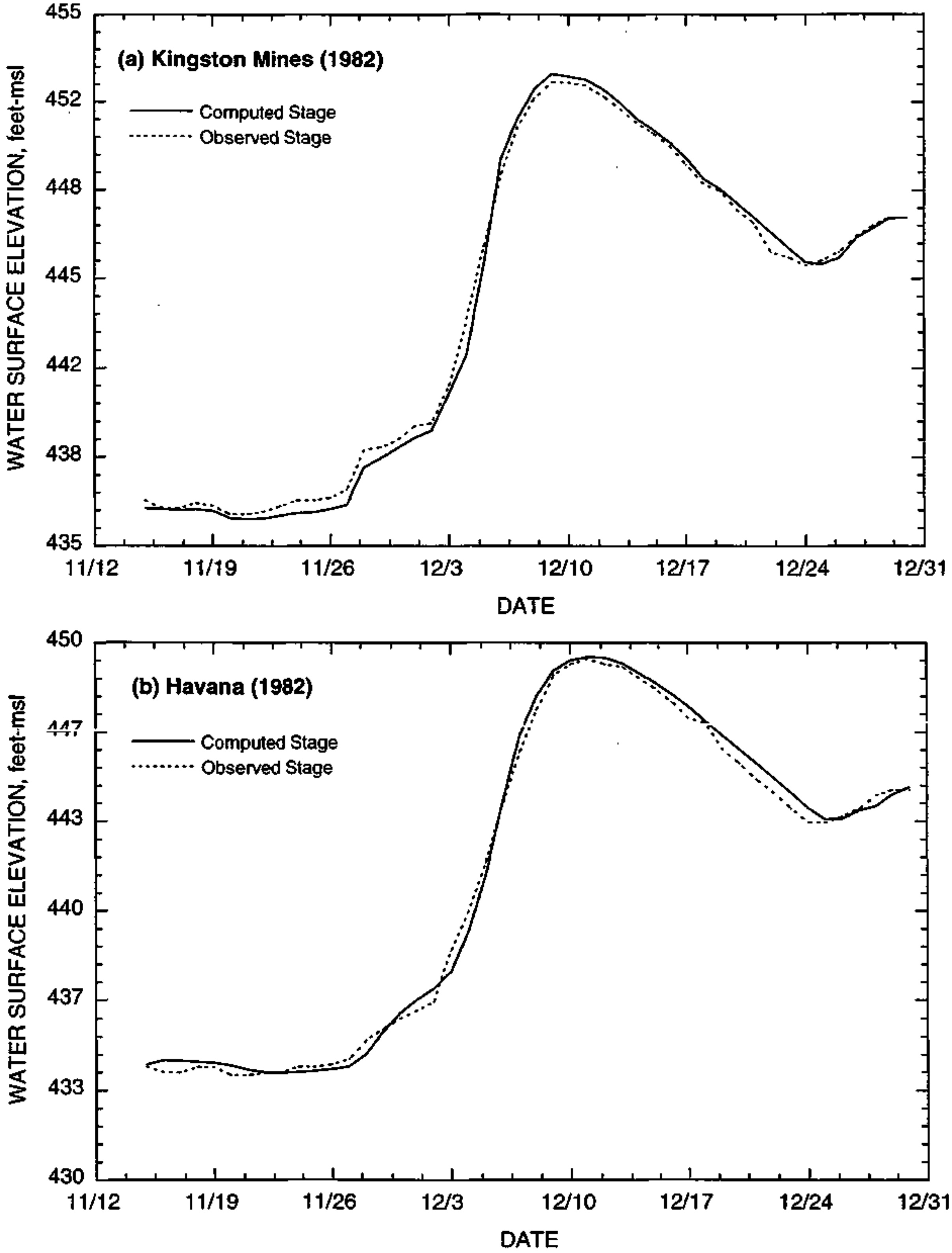
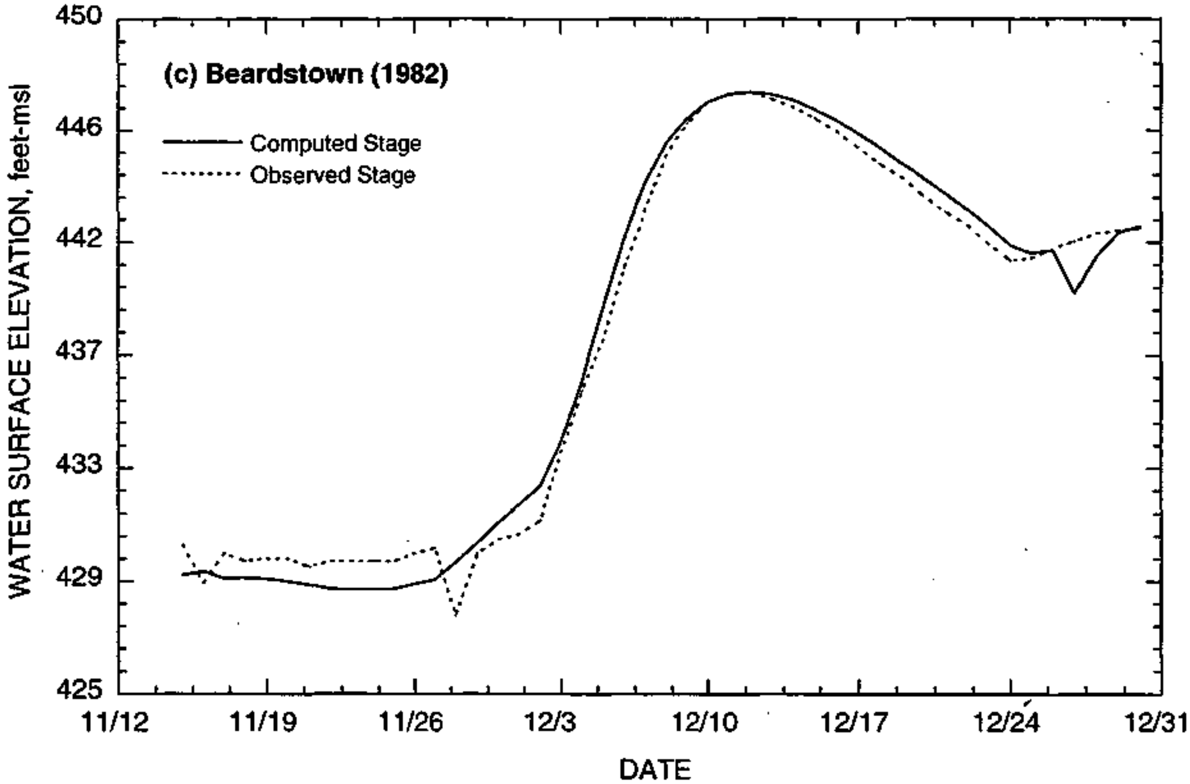


Figure 17. Water surface elevation hydrographs computed (two-reach model) and observed at (a) Kingston Mines, (b) Havana, (c) Beardstown, (d) Meredosia, (e) Valley City, (f) Florence, (g) Pearl, and (h) Hardin.



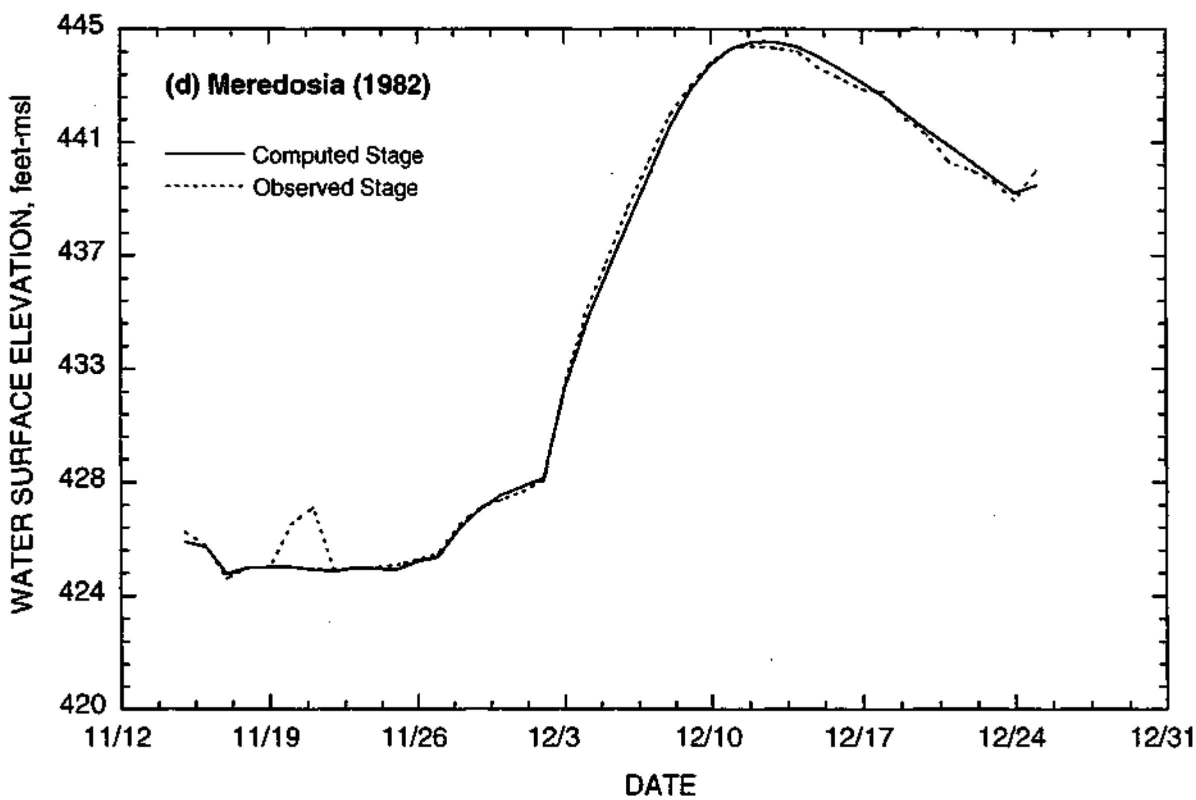


Figure 17. (continued)

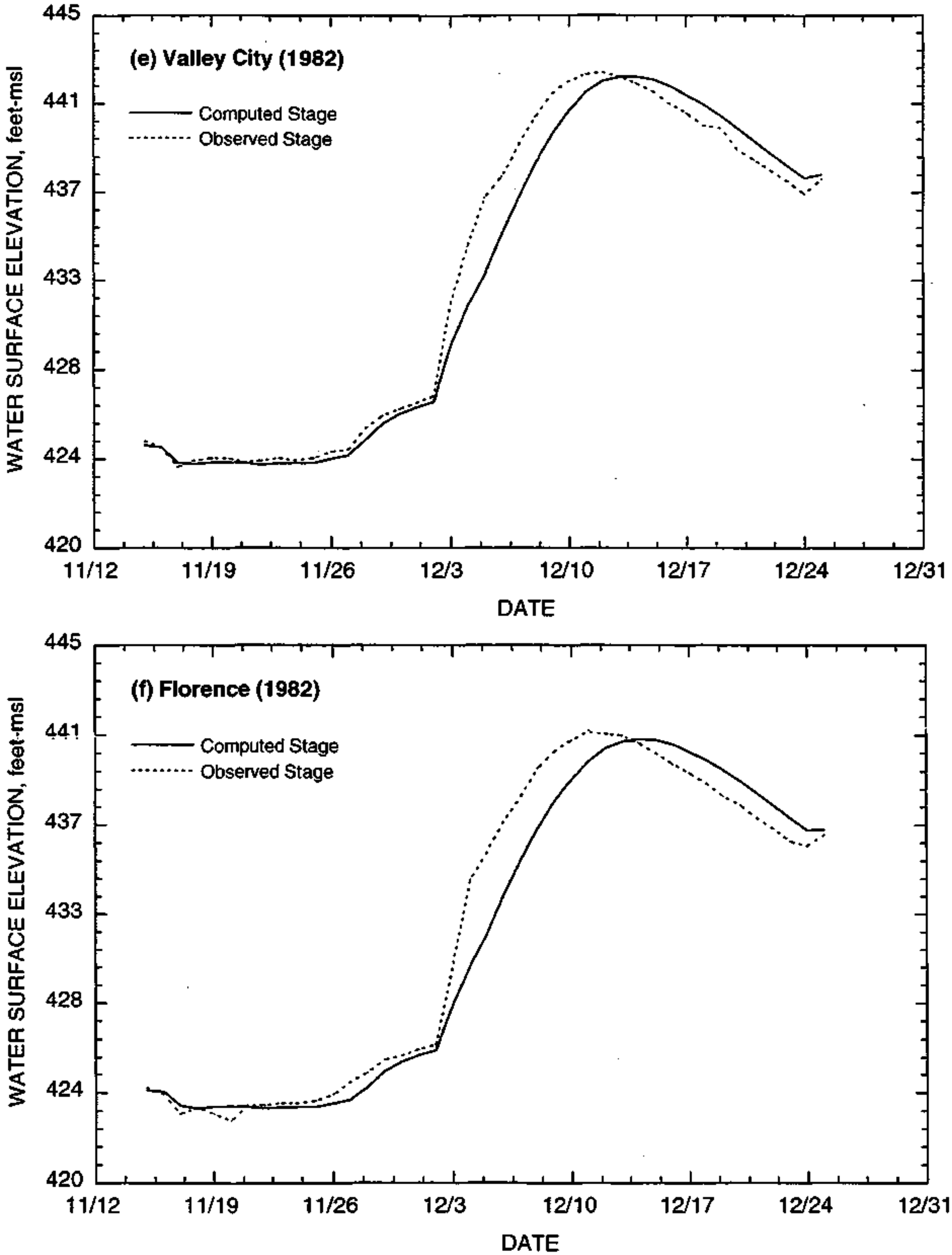
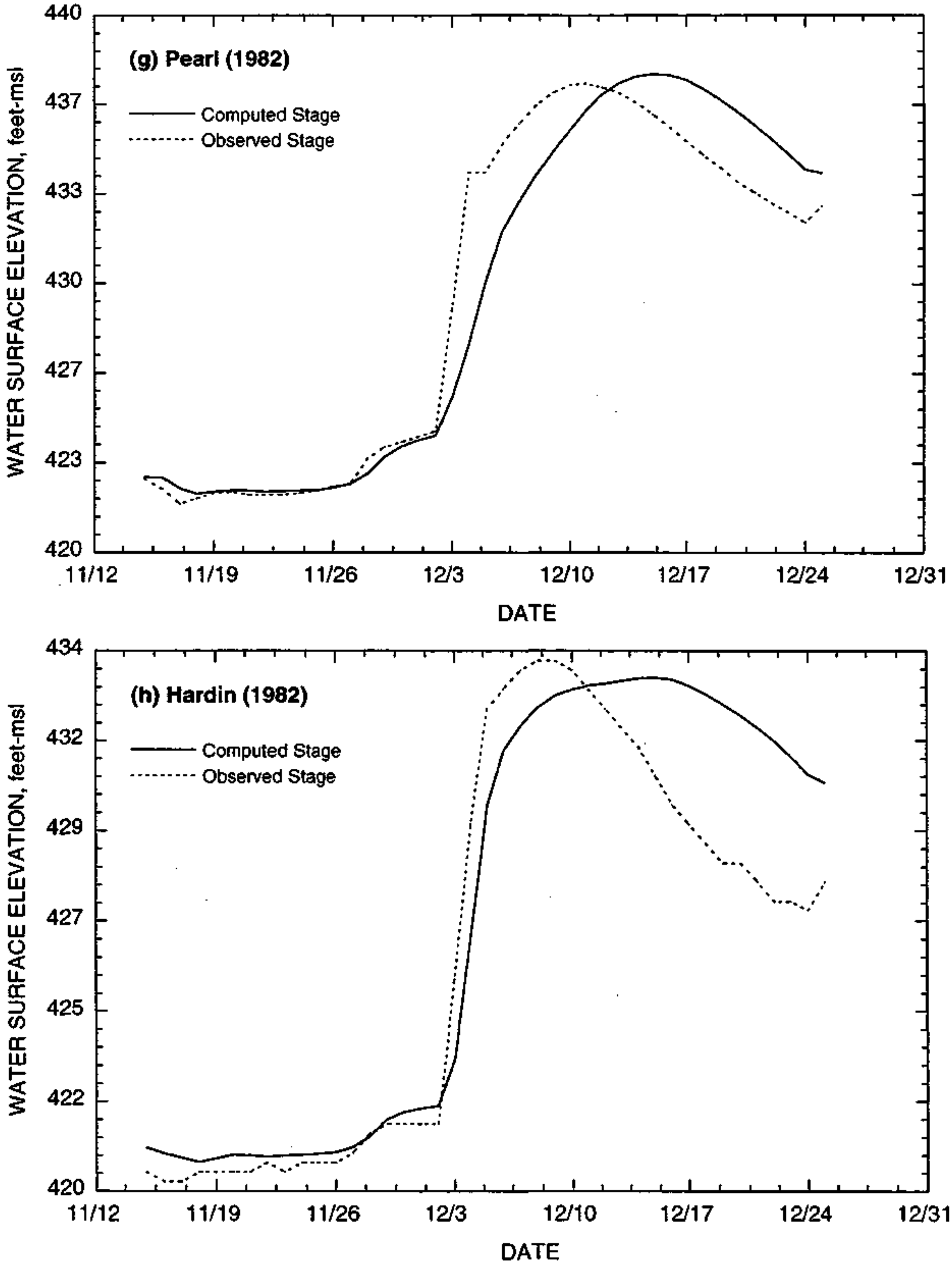


Figure 17. (continued)



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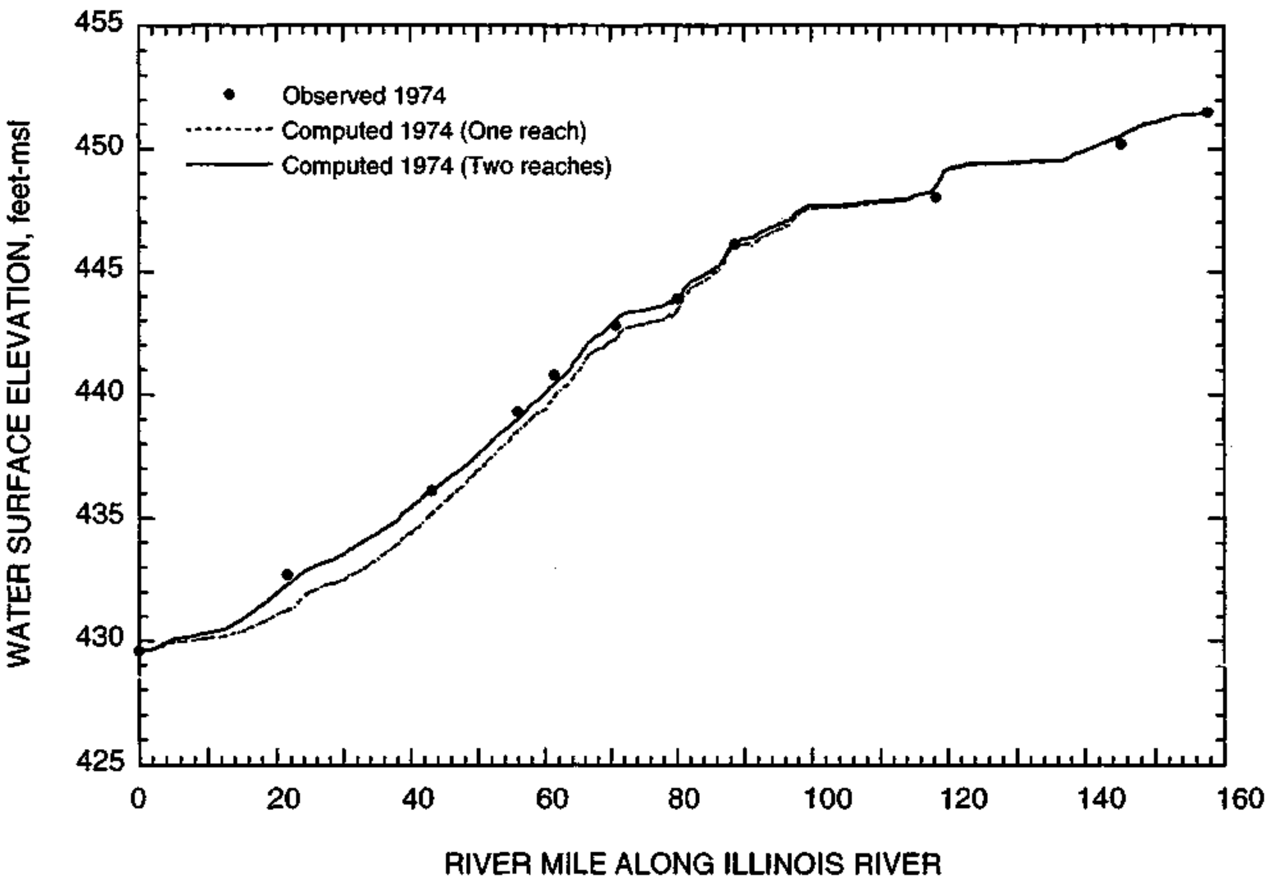


Figure 18. Comparison of peak water surface elevation profiles for one-reach and two-reach models with observed peak stages for the 1974 flood at gaging stations along the lower Illinois River

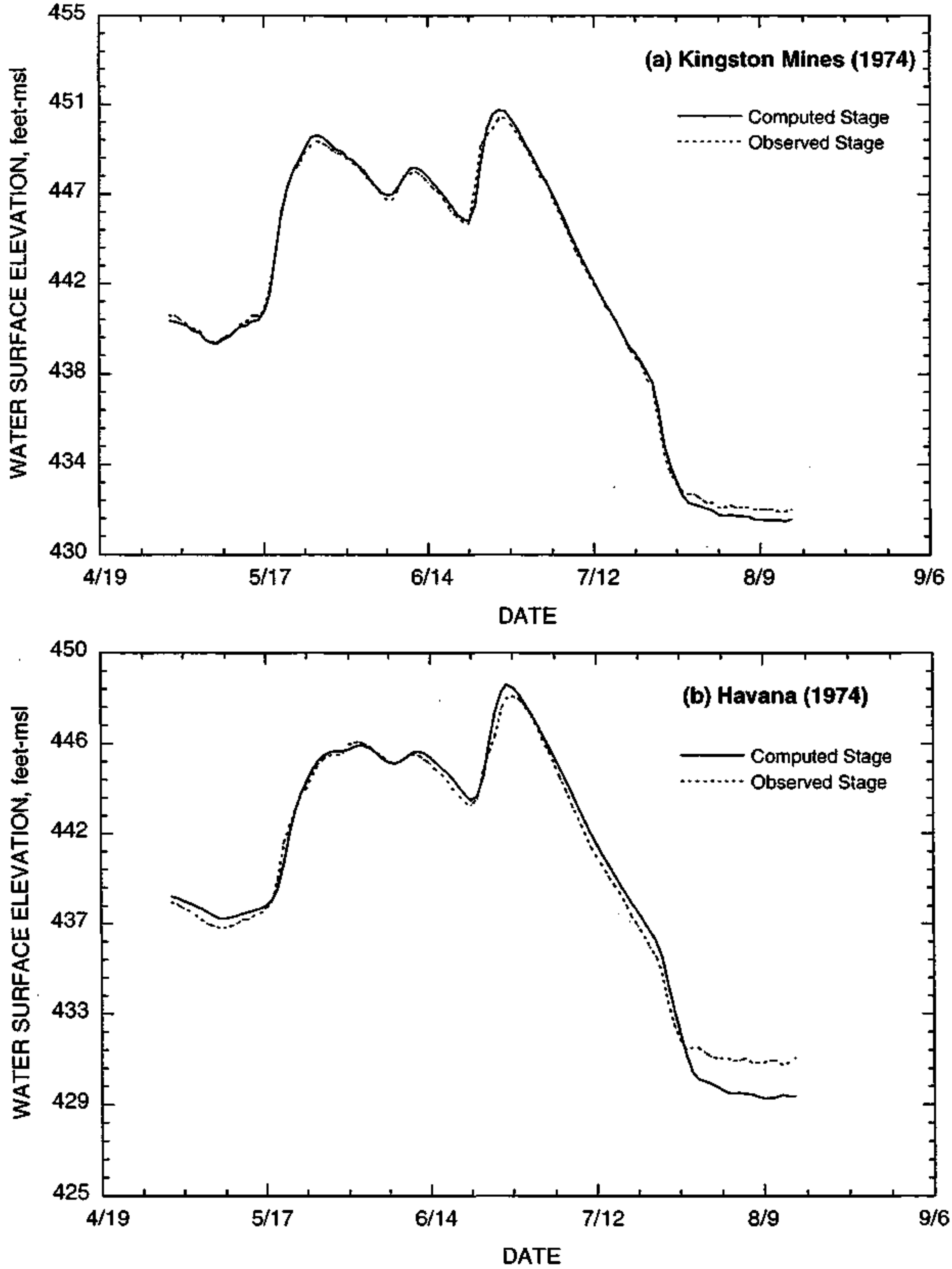


Figure 19. Water surface elevation hydrographs computed (two-reach model) and observed at (a) Kingston Mines, (b) Havana, (c) Beardstown, (d) Meredosia, (e) Valley City, (f) Florence, (g) Pearl, and (h) Hardin

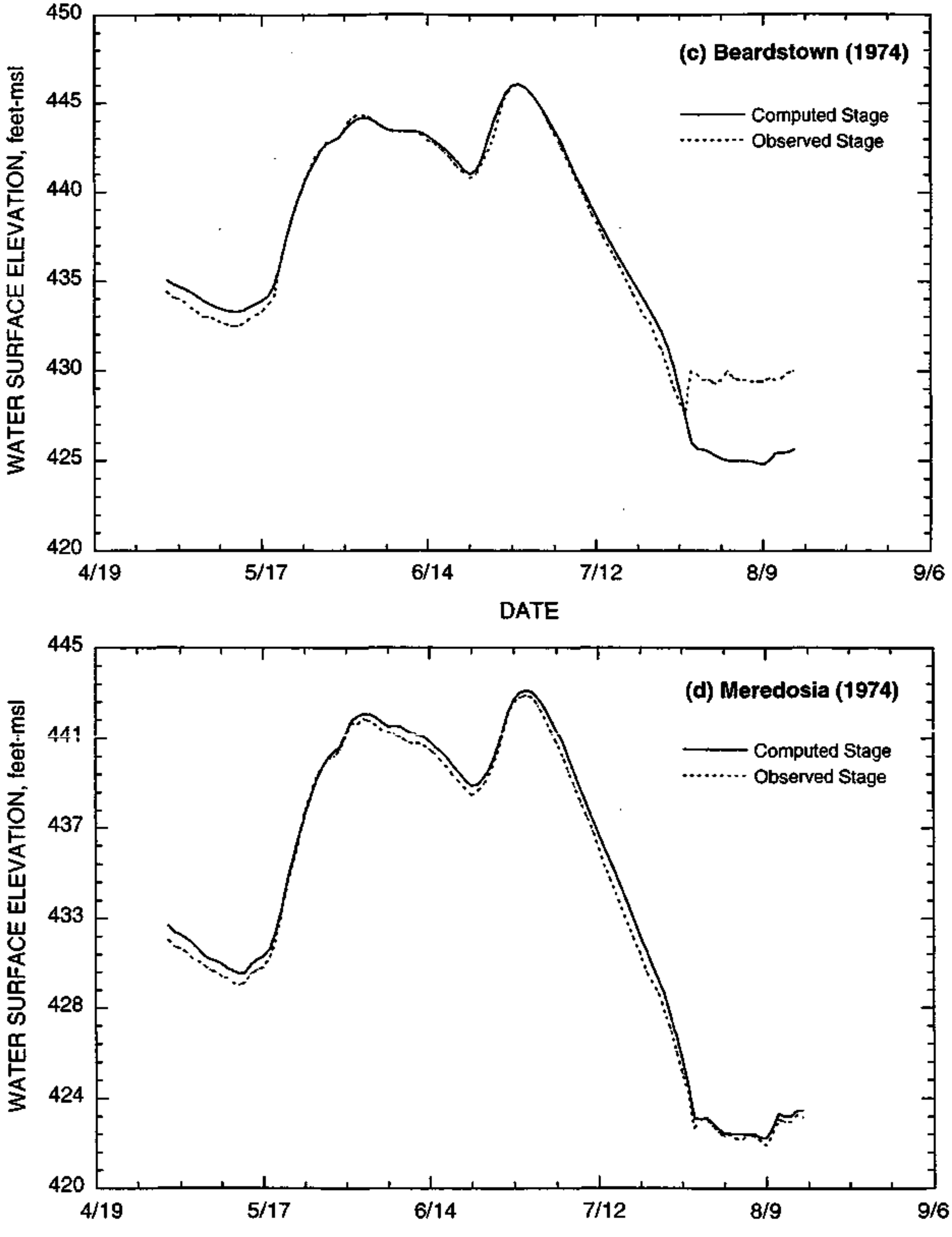
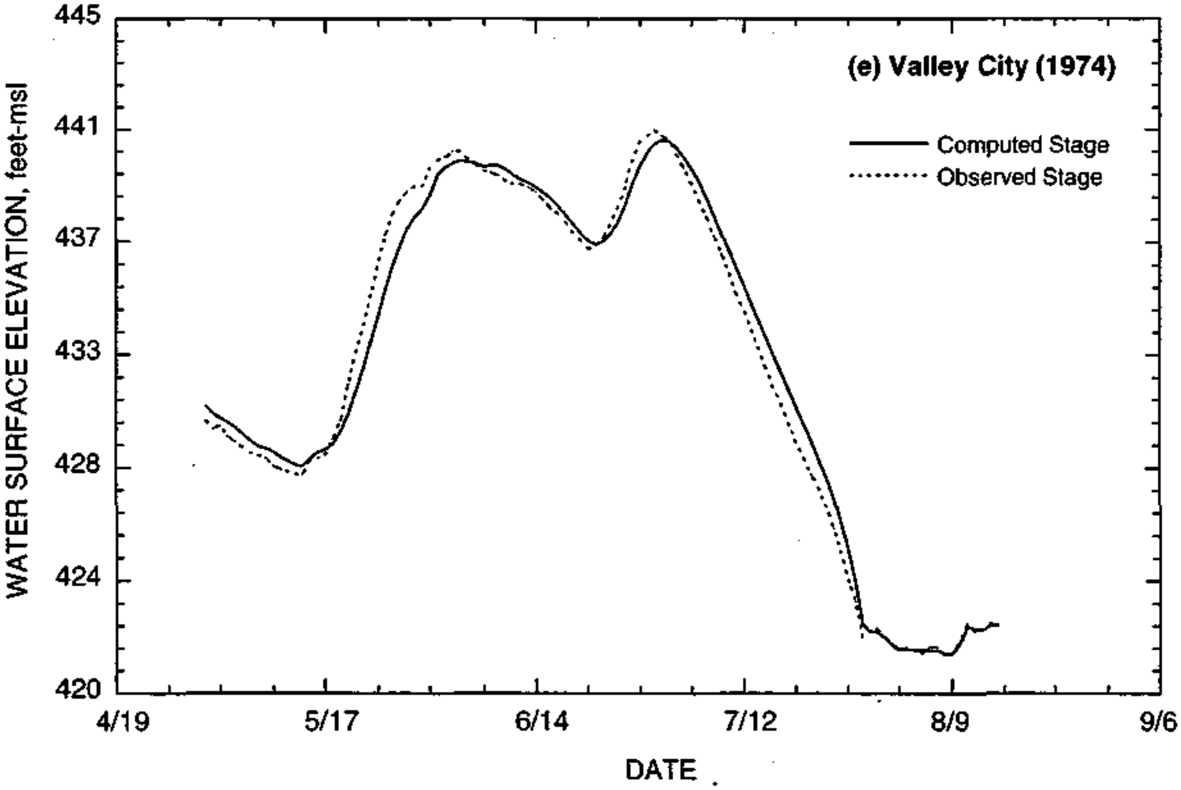


Figure 19. (continued)



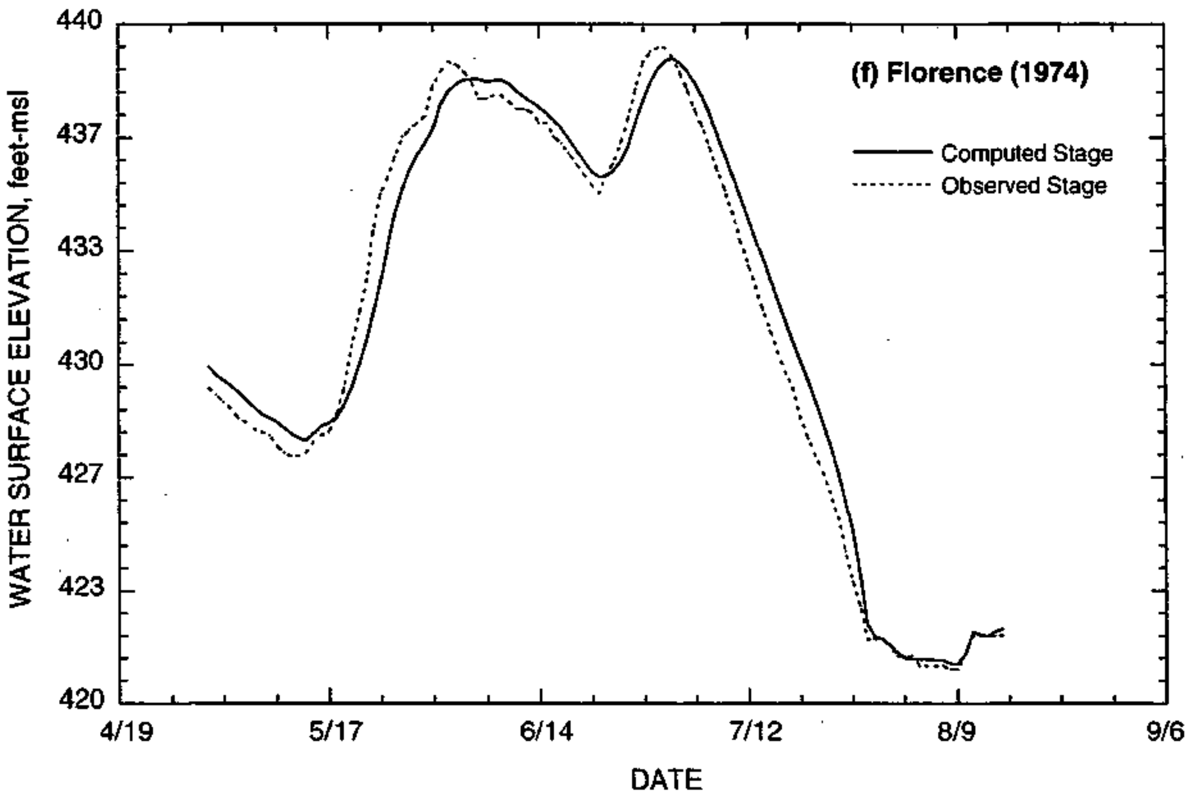
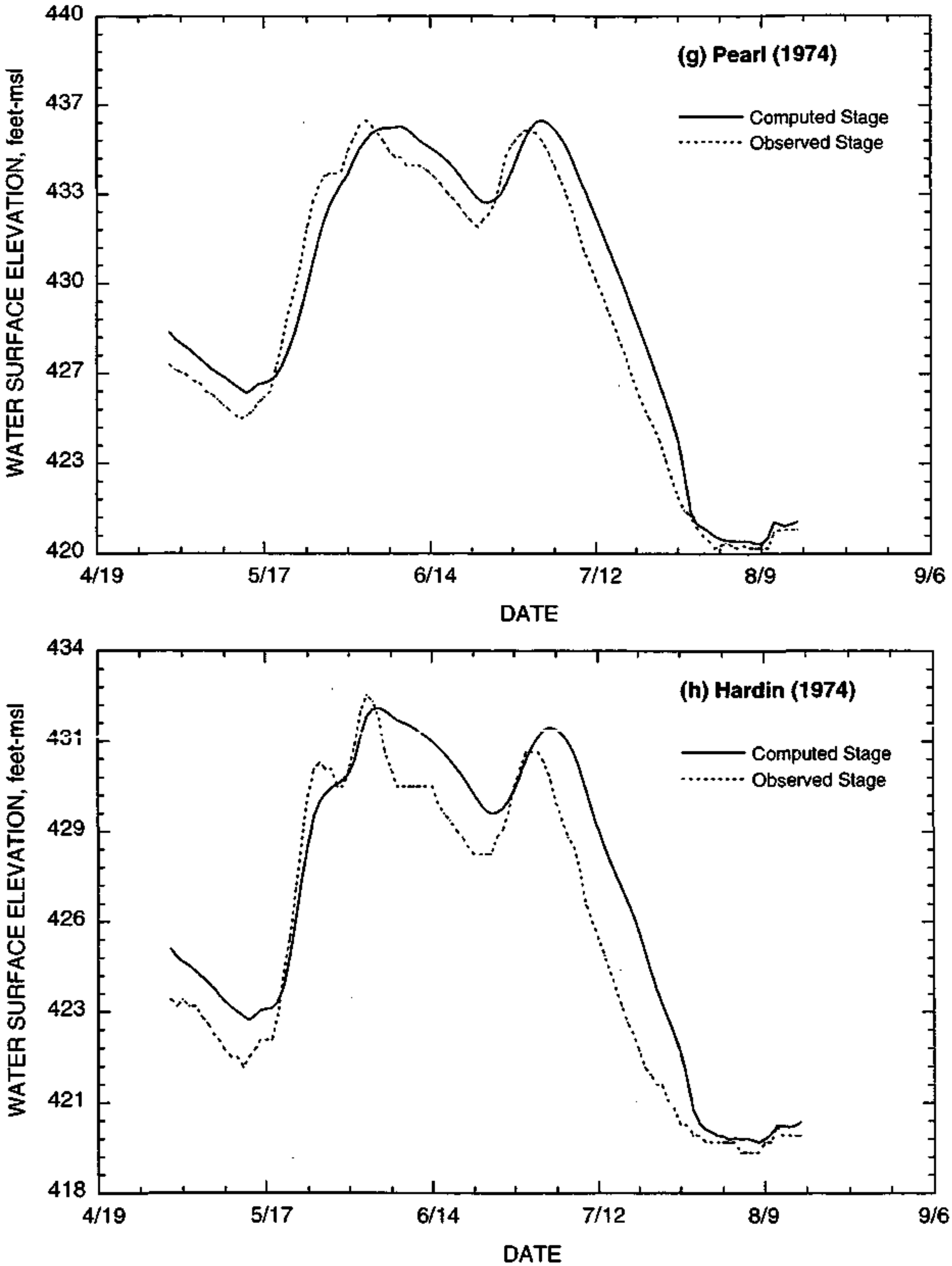


Figure 19. (continued)



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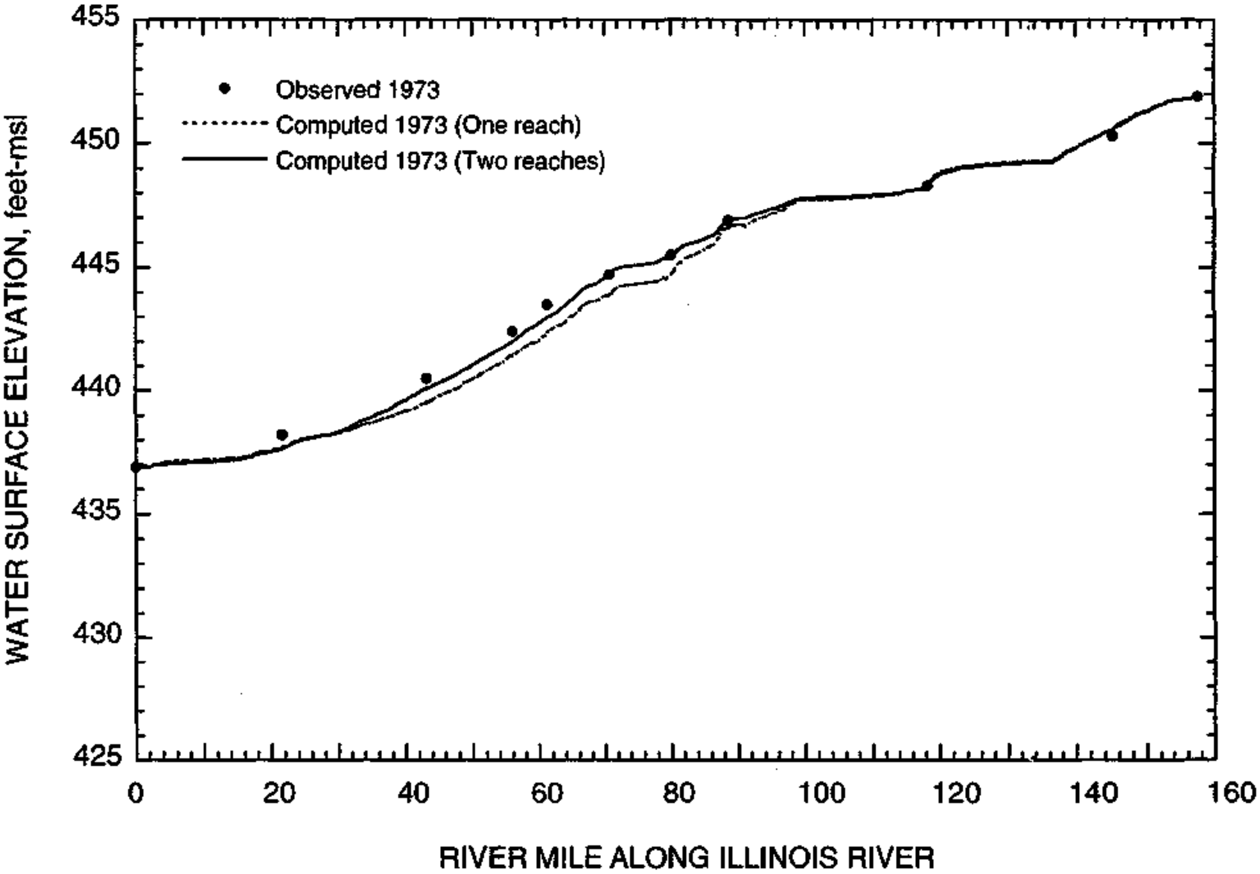


Figure 20. Comparison of peak water surface elevation profiles for one-reach and two-reach models with observed peak stages for the 1973 flood at gaging stations along the lower Illinois River

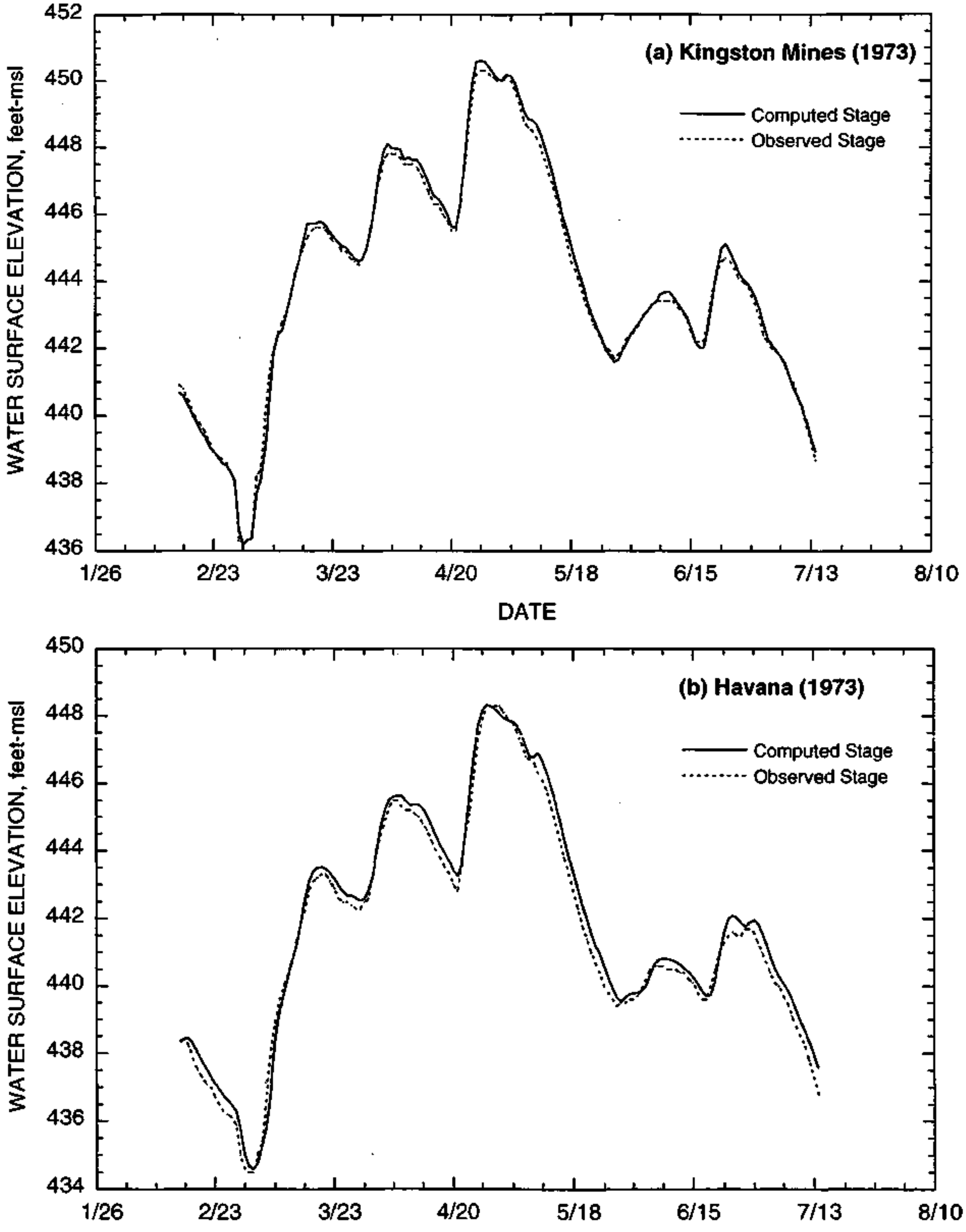
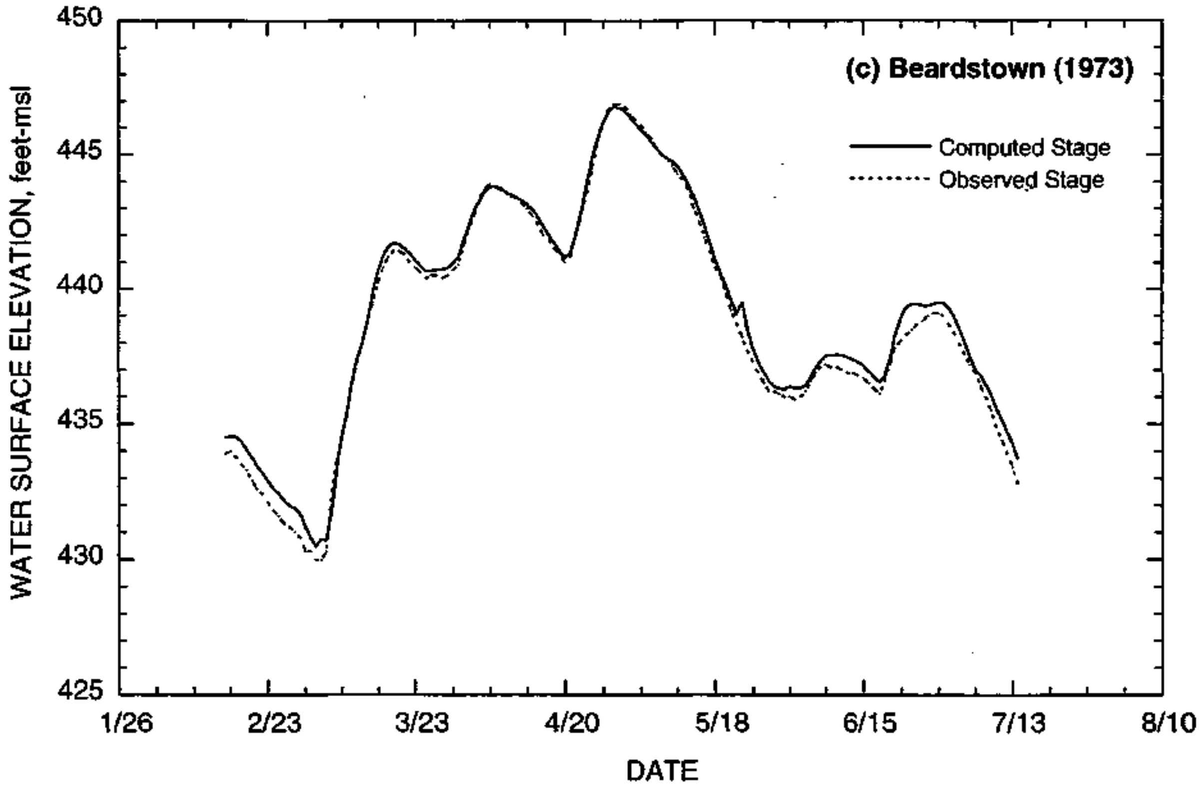


Figure 21. Water surface elevation hydrographs computed (two-reach model) and observed at (a) Kingston Mines, (b) Havana, (c) Beardstown, (d) Meredosia, (e) Valley City, (f) Florence, (g) Pearl, and (h) Hardin.



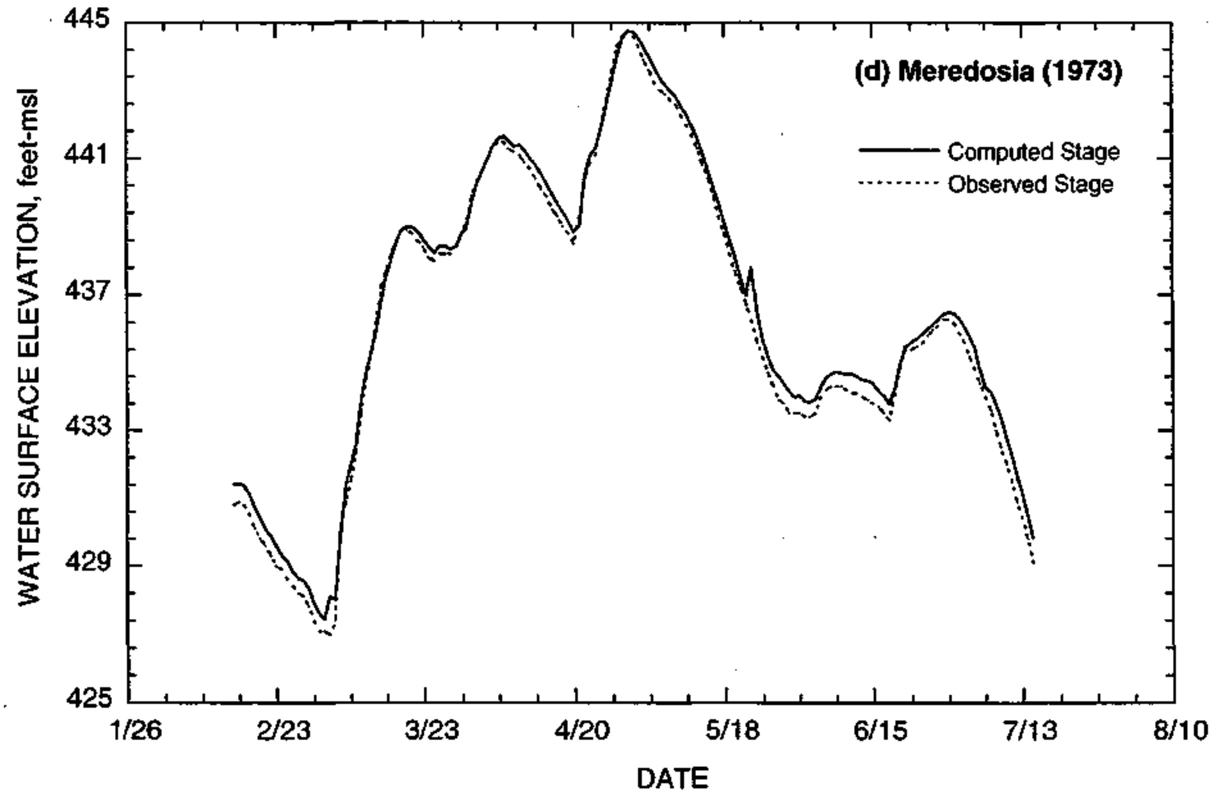
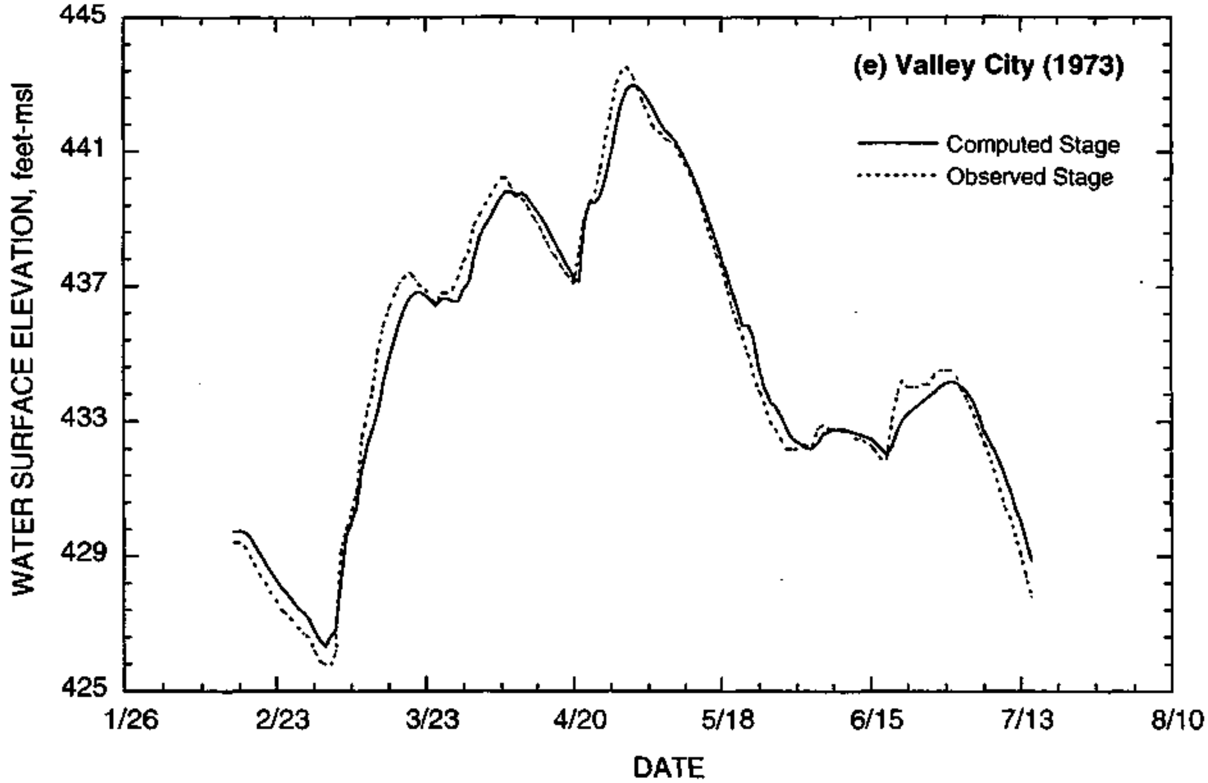


Figure 21. (continued)



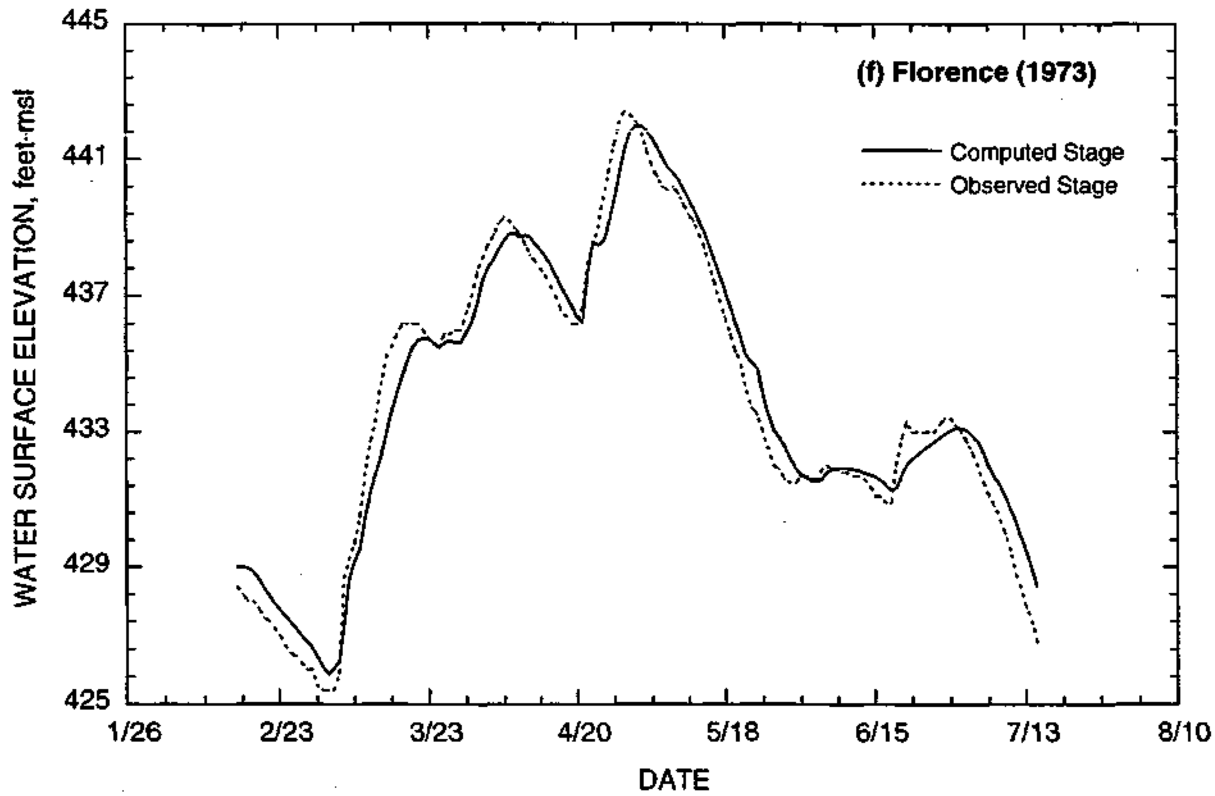
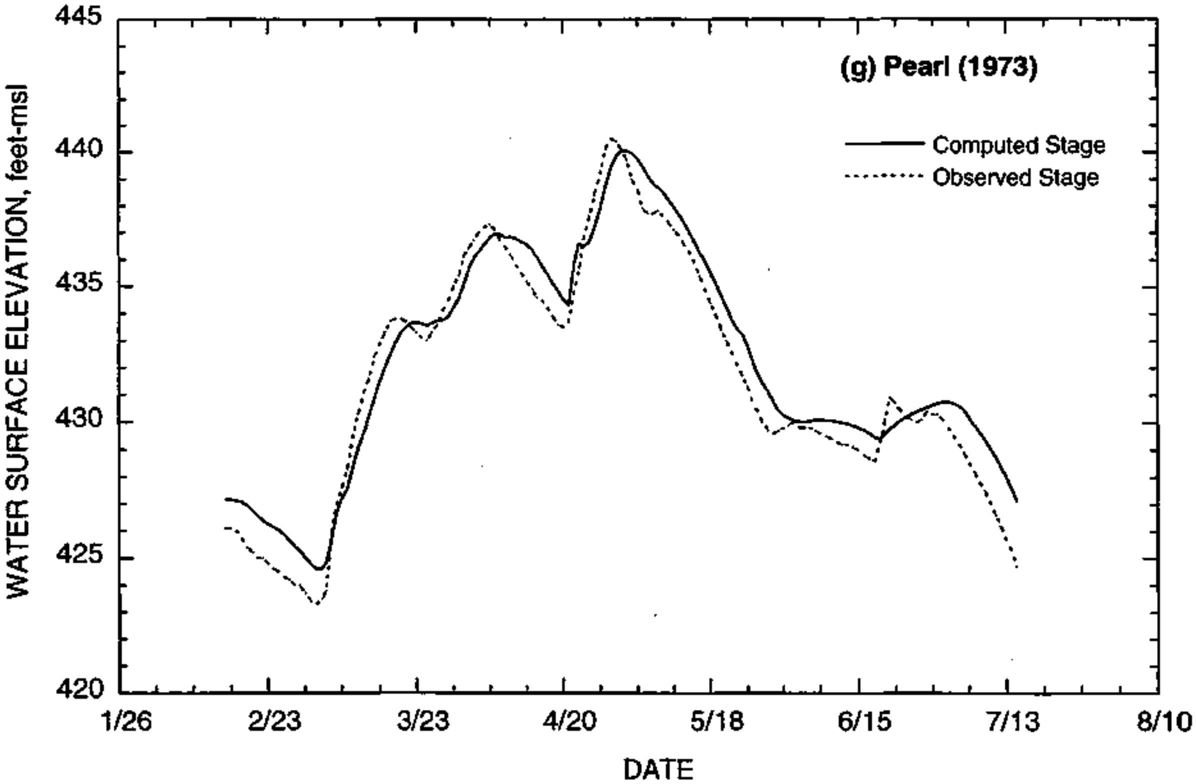


Figure 21. (continued)



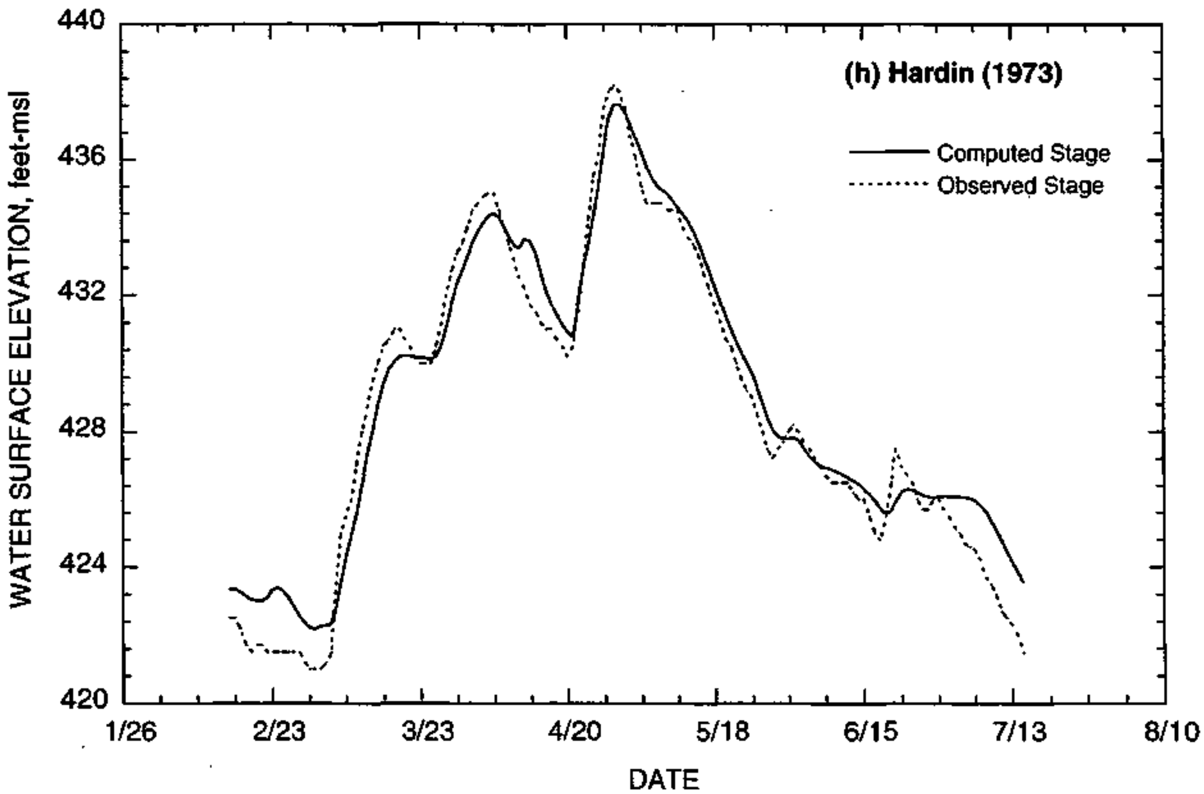


Figure 21. (concluded)

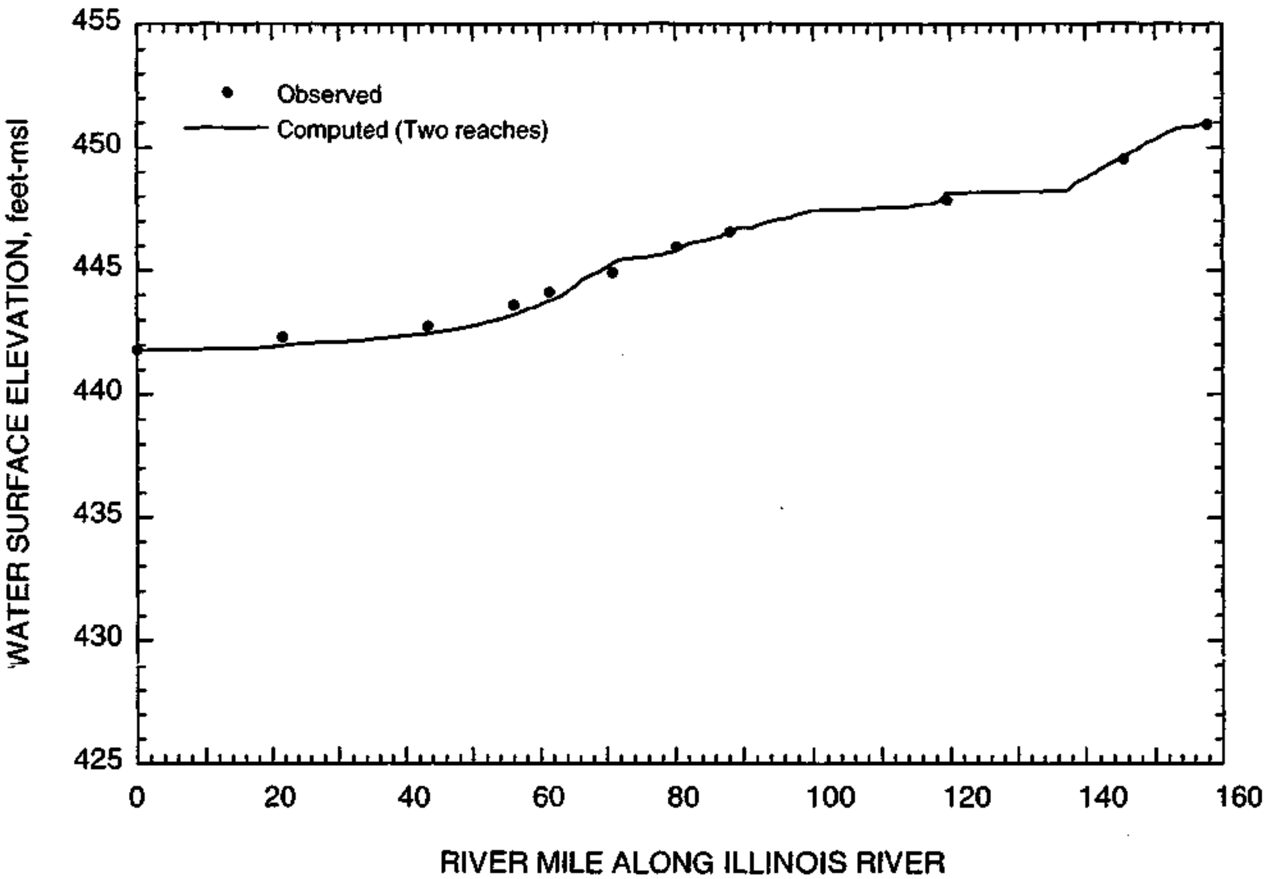


Figure 22. Comparison of peak water surface elevation profiles for two-reach model with observed peak stages for the 1993 flood at gaging stations along the lower Illinois River

hydrographs still lagged the observed hydrographs by one to two days. The model also underpredicted the peak WSE at Hardin by about 0.5 feet, and the computed hydrograph has a broader crest in comparison to the observed record. The tail of the hydrograph is grossly overpredicted at this station by as much as 4 feet. The poor performance of the model for this flood may be due to the ice conditions on the Illinois River in December or the variation in magnitude and timing of the flow from Macoupin Creek.

Figures 18 and 19 show the simulation results for the June 1974 flood. In Figure 18, the accuracy of the model is observed to deteriorate downstream of La Grange L&D for the one-reach model, but the two-reach simulation results closely match the observed WSE at each of the stations. Figure 19 shows the two-reach WSE hydrographs at the stations. The computed hydrographs closely matched the observed hydrographs. The computed hydrographs, however, lagged the observed data by up to one day from Valley City to Hardin. The greatest difference in WSE is at Hardin, an observation also made in previous simulations.

The one-reach peak WSE for April 1973 is very poor in comparison to the two-reach simulations shown in Figure 20. The two-reach model significantly under-estimated the peak WSE downstream of Meredosia, but the errors are less than 0.25 feet at these stations. Figure 21 shows the computed and observed hydrographs for the nine interior stations for the two-reach model. The hydrographs are generally in phase at each station, but a difference of up to 0.75 feet was observed at Florence, Pearl, and Hardin.

The flood peak of 51,200 cfs, recorded at Kingston Mines on July 29, 1993, has a return interval of approximately three years for a 53-year (1941-1993) annual peak flow record. A flood peak of 89,630 cfs was observed at Meredosia on August 1, 1993, and this discharge has an estimated return period of about five years for the same length of record. The low return periods of the flood peaks at these two stations indicate that the peak stages on the lower Illinois River in July and August of 1993 were not produced by a major flood on the river. Instead, the prolonged record stages in the Mississippi River raised the stages along the Illinois River to levels much higher than the corresponding 3- to 5-year return period flows.

The extent of the influence of the backwater effect can be determined by examining the peak stages at Illinois River stations upstream of Grafton. The peak stage at Grafton was 441.8 feet-msl, which is the highest in the 53-year record. The peak stage at Meredosia, which is 70.8 miles upstream, was 3.10 feet higher than the peak stage at Grafton. This difference in WSEs between the two stations is the lowest when compared with the March 1985, April 1979, and December 1982 floods where the differences in WSE were at least four times higher (15.13, 13.93, and 12.80 feet, respectively). It is, therefore, apparent that the backwater effect extended up to Meredosia and probably as far upstream as Kingston Mines.

By ranking the annual peak WSE (1941-1993) for the stations between Meredosia and Kingston Mines, the ranks for the 1993 flood stages are:

		Peak WSE		
Station	*River Mile*	*(ft-msl)*	*Date*	*Rank*
Grafton	0.0	441.8	8/1/93	1
Meredosia	70.8	444.9	7/28/93	4
La Grange	80.2	445.95	7/27/93	5
Beardstown	88.1	446.6	7/28/93	6
Havana	119.6	447.9	7/29/93	7
Kingston Mines	145.6	448.4	7/30/93	14

The rank at Meredosia is 4 and the rank increased by one for consecutive upstream stations up to Havana. At Kingston Mines the rank for the peak stage jumped to 14. From the listing in the table above, there is an increase in rank of 6 between Grafton and Havana (119.6 miles) and an increase of 7 between Havana and Kingston Mines (26.0 miles). The sharp jump in rank over the short distance between Havana and Kingston Mines indicates that the backwater effect may not have reached as far as Kingston Mines. The rank of 14 at Kingston Mines corresponds to about a four-year reccurrence interval, which is about the same as for the peak flood at that station. The influence of the rising stages on the Mississippi seemed to end somewhere between Havana (R.M. 119.6) and Kingston Mines (R.M. 145.6).

Since the backwater effect resulting from the Mississippi River flood extended upstream of Havana, the UNET model validation of the 1993 flood was carried out for both the Alton and La Grange Pools. Figure 22 shows the computed peak WSE profile and the observed data. The figure shows that peak WSEs were somewhat underpredicted at all stations in the Alton Pool except Valley City. However, the predicted WSEs are all within 0.5 feet of the observed peaks.

PEAK STAGE REDUCTION FROM LEVEE STORAGE OPTIONS

This chapter outlines the next phase of the Managed Flood Storage Options project. This phase is currently being implemented and will be reported in the third project report. Current tasks include the determination of flood profiles for 10-, 25-, 50-, and 100-year return periods; examination of the variability of flow in the Illinois River and flood peak timing from Illinois River tributaries; and evaluation of flood reduction due to conversion of some of the levees and drainage districts in the Alton and La Grange Pools to the managed flood storage option.

Recurrence Interval Flood Profiles

As stated in previous chapters, the simulation of flood elevation profiles along the lower Illinois River requires the selection of flood hydrographs for the tributaries and stage hydrographs for the upstream and downstream boundaries on the main river stem. For instance, the selection of the appropriate tributary flow hydrographs that will generate the 100-year peak WSE profile along the Illinois River is a major task in the modeling process. This is because various combinations of tributary flows can, theoretically, be applied to generate the 100-year flood elevation profile in different reaches of the river. The problem becomes more complex because a 100-year flood elevation profile is generally not generated by a 100-year flood flow in the Illinois River. There is no direct relationship between stage and discharge, so the 100-year flood elevation may be generated by more than one flow at each gaging station. This has already been observed in some of the historical floods on the Illinois River. For instance, in December 1982, the third highest flood of 112,000 cubic feet per second or cfs occurred at Meredosia and the highest flood of 88,000 cfs at Kingston Mines in the concurrent record from 1941-1993. In April 1979, a flood of 111,000 cfs occurred at Meredosia and 72,000 cfs at Kingston Mines ranked fourth and sixth at these stations, respectively. It would be expected that the 1982 WSE profile will be higher than the 1979 profile. However, from the plot of the peak WSE at the gaging stations along the lower Illinois River (Figure 23), the 1979

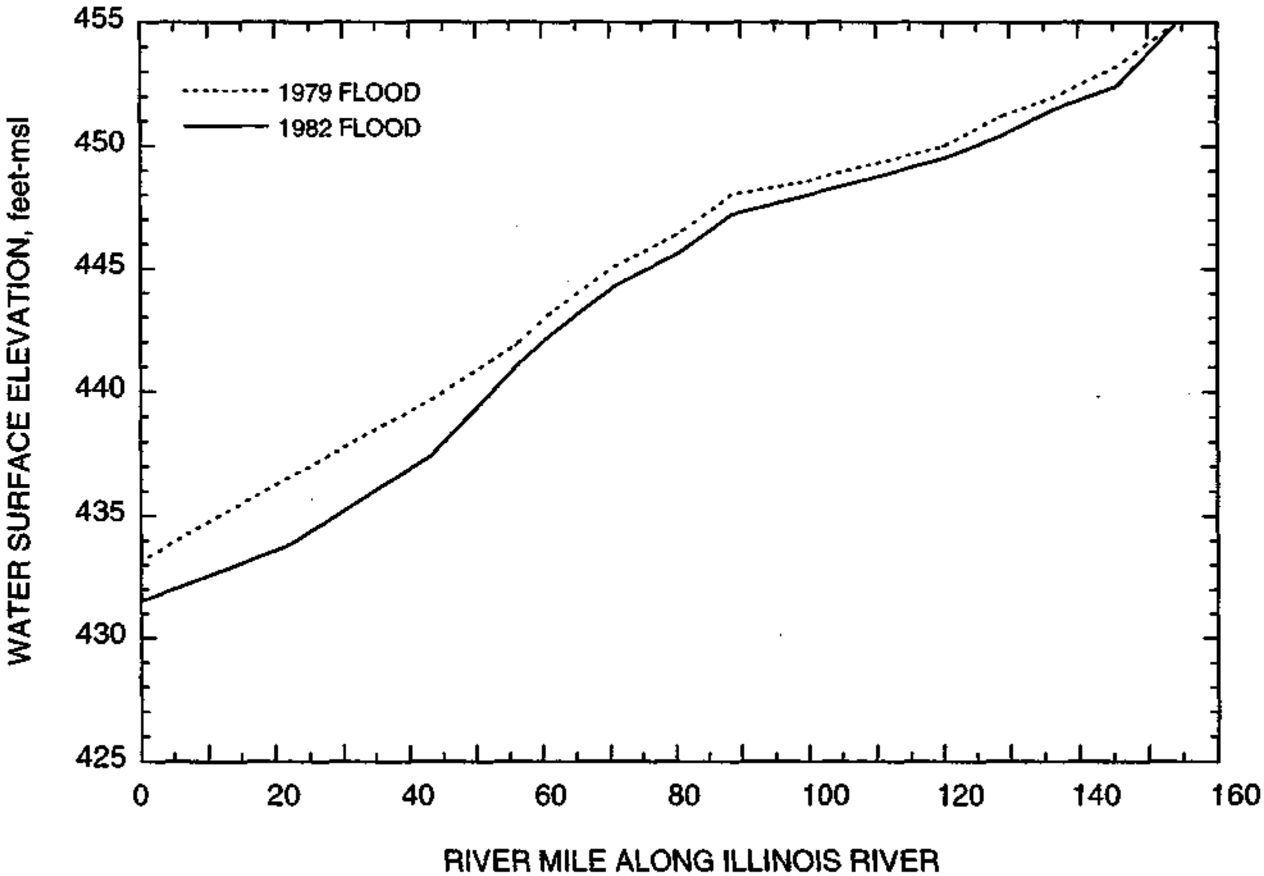


Figure 23. Observed peak water surface elevation profiles for the April 1979 and December 1982 floods on the lower Illinois River

WSEs are higher than the 1982 elevations. The approach adopted in this project is described in the next few paragraphs.

Using the flood frequency analysis described in the first project report (Singh, 1996), the 2-, 10-, 25-, 50-, 100-, and 500-year floods were evaluated for the five stations that represent the ungaged tributaries (Big Bureau, Big, Spring, Hadley, and Bay Creeks). Table 9 shows the results of these analyses. In addition, the table contains the frequency analysis results for the five gaged tributaries (Mackinaw, Spoon, Sangamon, and La Moine Rivers, and Macoupin Creek) on the lower Illinois River.

The first six top floods at each of the ten stations were selected and the recorded discharges for ten days preceding and for ten days succeeding the occurrence of the peak discharge were plotted for each flood. Each of the six hydrographs was first normalized with the corresponding peak discharge and then plotted on the same 20-day base with the peaks occurring on the same day as shown in Figures 24-33. A representative normalized hydrograph for a station was then obtained from the six hydrographs. As much as possible, the representative hydrographs were drawn as closely as possible to the first three top-flood hydrographs. The synthesized station hydrographs that will be used in the simulation for a return period were obtained by multiplying the ordinates of the representative normalized hydrograph by the corresponding peak flow from Table 9. The synthesized hydrographs for the various return periods were read into the Data Storage System database and will be for each of the tributaries.

The next step was to provide the appropriate time lag for each of the synthetic hydrographs by examining the timing of the peak floods for the flood events in 1943, 1974, 1979, 1982, and 1985 used for model calibration and validation. The time lag of the peaks of the recorded hydrographs at the tributary gages relative to Meredosia (Table 10) seem to be between six and seven days. Peoria L&D seems to be ahead of Meredosia by about five days, suggesting that the upstream boundary hydrograph at Peoria L&D will lag behind the tributary station hydrographs by one to two days. One of the series of simulations that will be carried out will involve the variation of the lags so that the flood peak from some of the tributaries coincides with the flood wave on the Illinois River. This

Table 9. Tributary Flood Peaks (cfs) at Various Recurrence Intervals

Tributary	*2-year*	*10-year*	*25-year*	*50-year*	*100-year*	*500-year*
Mackinaw River	10,300	27,600	39,600	50,000	62,000	97,800
Spoon River	14,000	26,100	32,300	37,800	41,000	51,800
Sangamon River	24,500	46,300	59,000	72,000	87,500	138,000
LaMoine River	12,000	23,000	26,800	31,000	35,000	45,000
Macoupin Creek	10,800	22,800	29,300	34,600	39,600	59,400
Big Bureau Creek	4,210	8,100	10,000	11,500	12,800	15,500
Big Creek	773	1,157	1,336	1,465	1,592	1,882
Spring Creek	1,840	5,750	8,300	10,500	13,200	20,000
Hadley Creek	7,339	12,650	15,240	17,337	19,811	25,162
Bay Creek	5,134	10,541	13,210	15,291	17,491	23,499

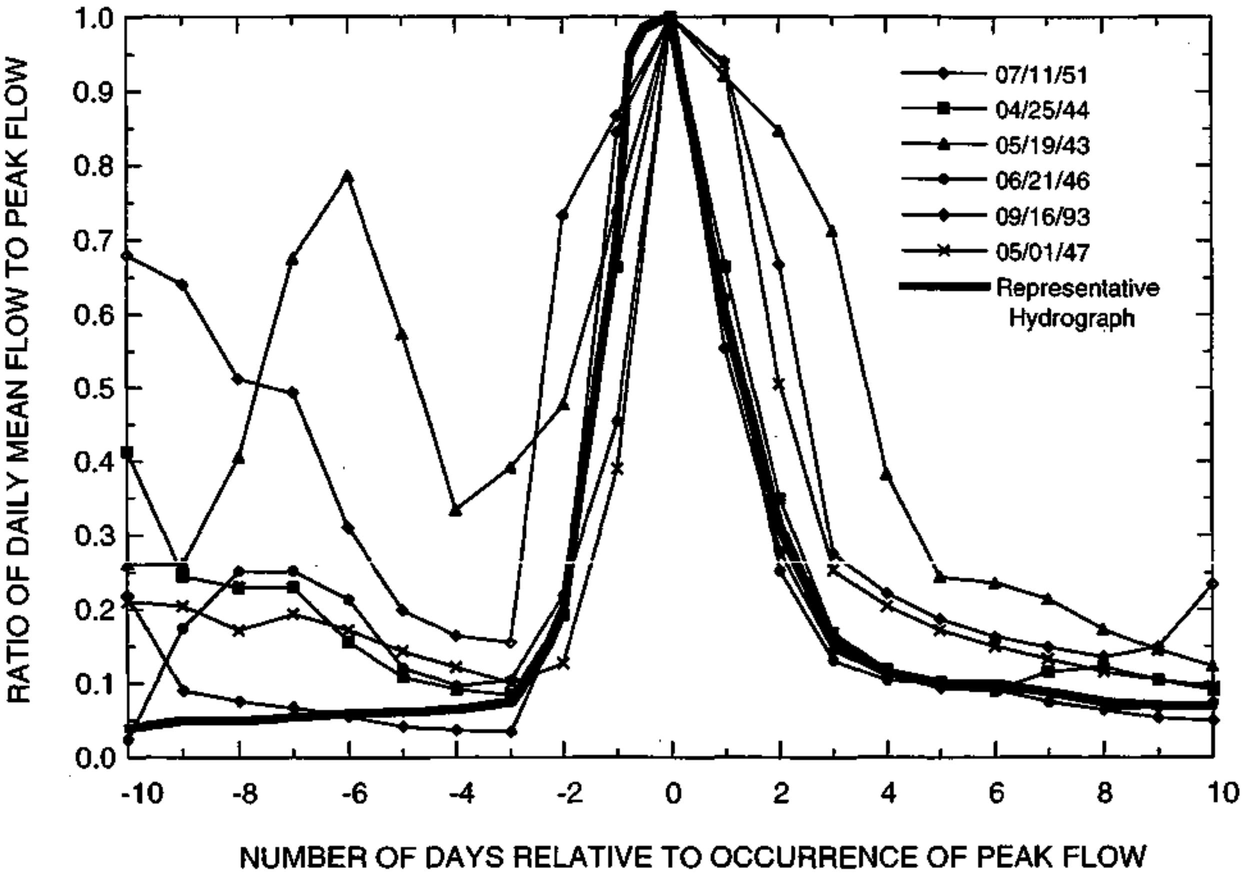


Figure 24. Normalization of the hydrographs of the top six floods, recorded at the Mackinaw River gage near Green Valley, by their respective peak flows and selection of a representative discharge hydrograph

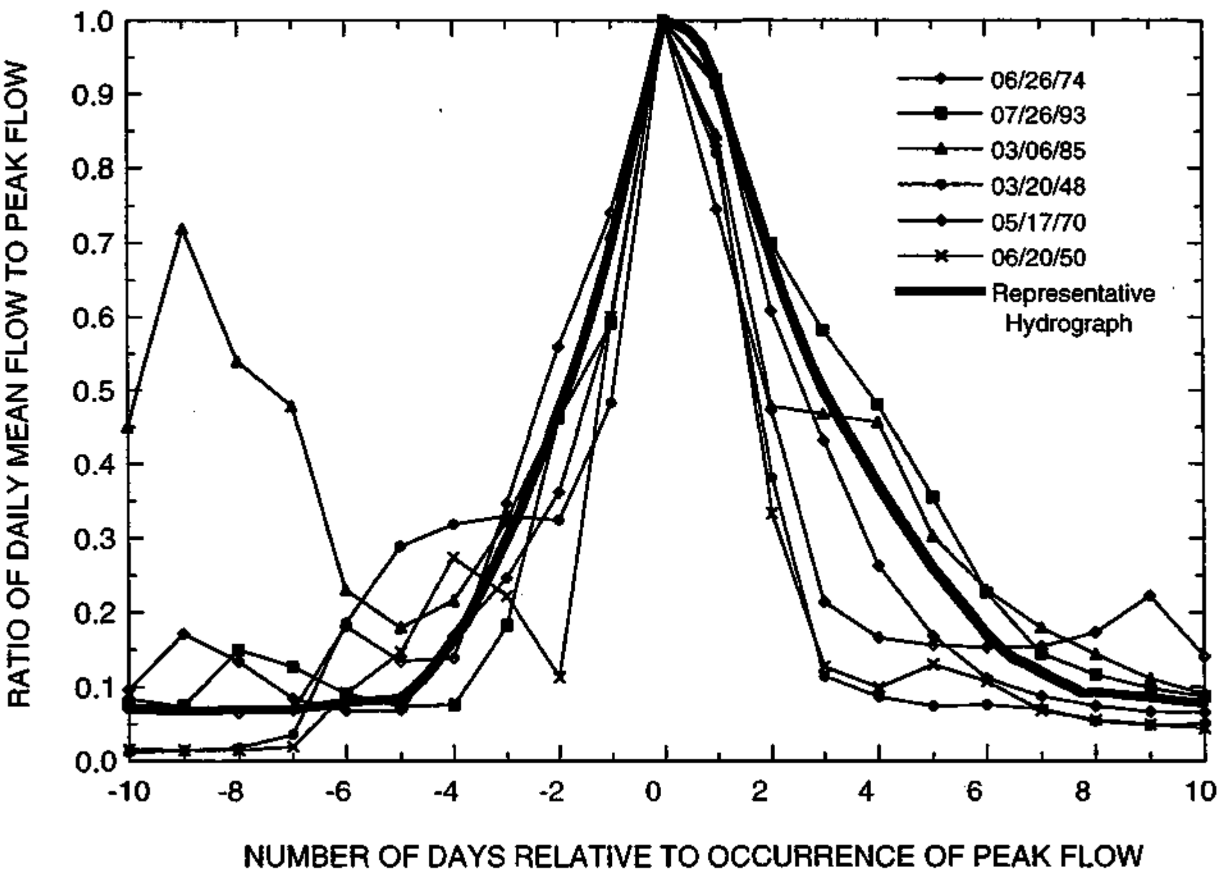


Figure 25. Normalization of the hydrographs of the top six floods, recorded at the Spoon River gage at Seville, by their respective peak flows and selection of a representative discharge hydrograph

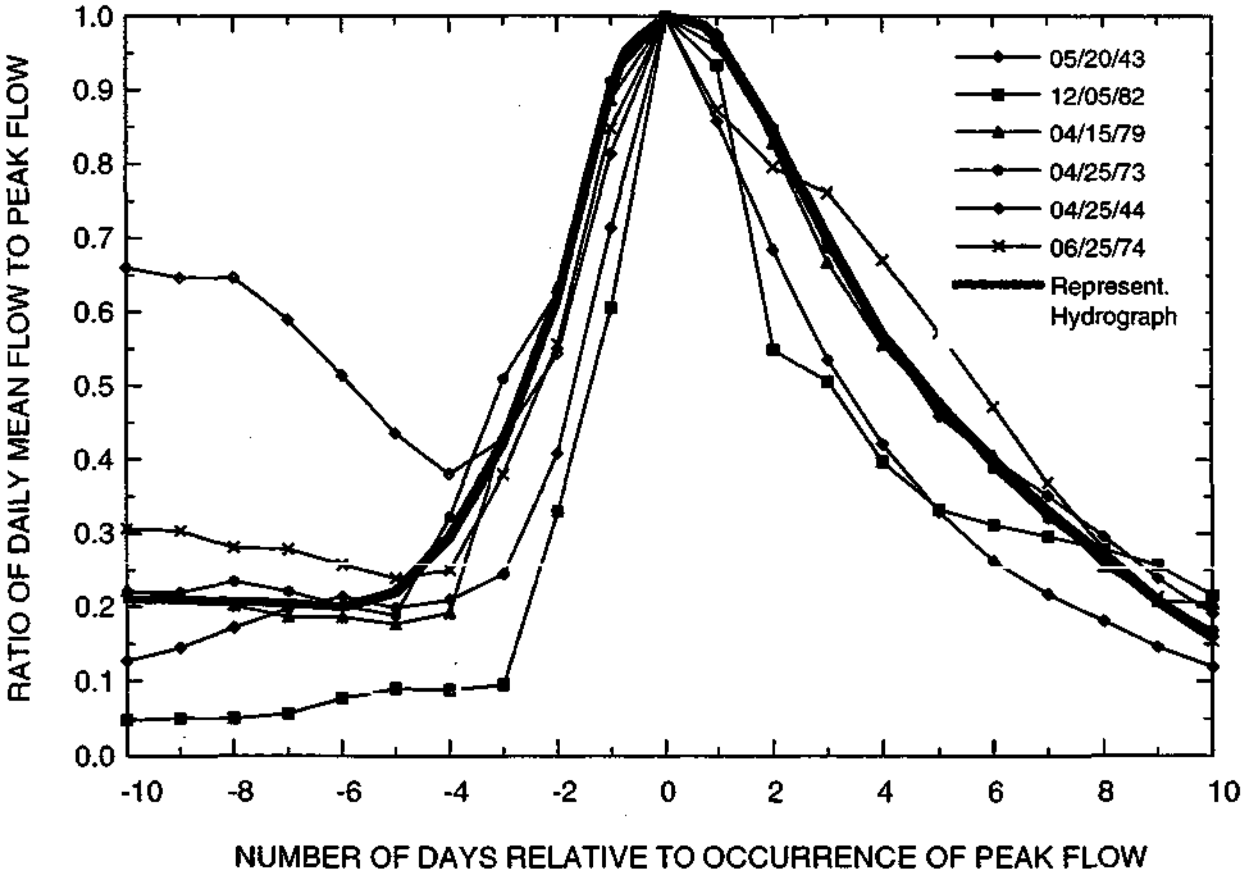


Figure 26. Normalization of the hydrographs of the top six floods, recorded at the Sangamon River gage near Oakford, by their respective peak flows and selection of a representative discharge hydrograph

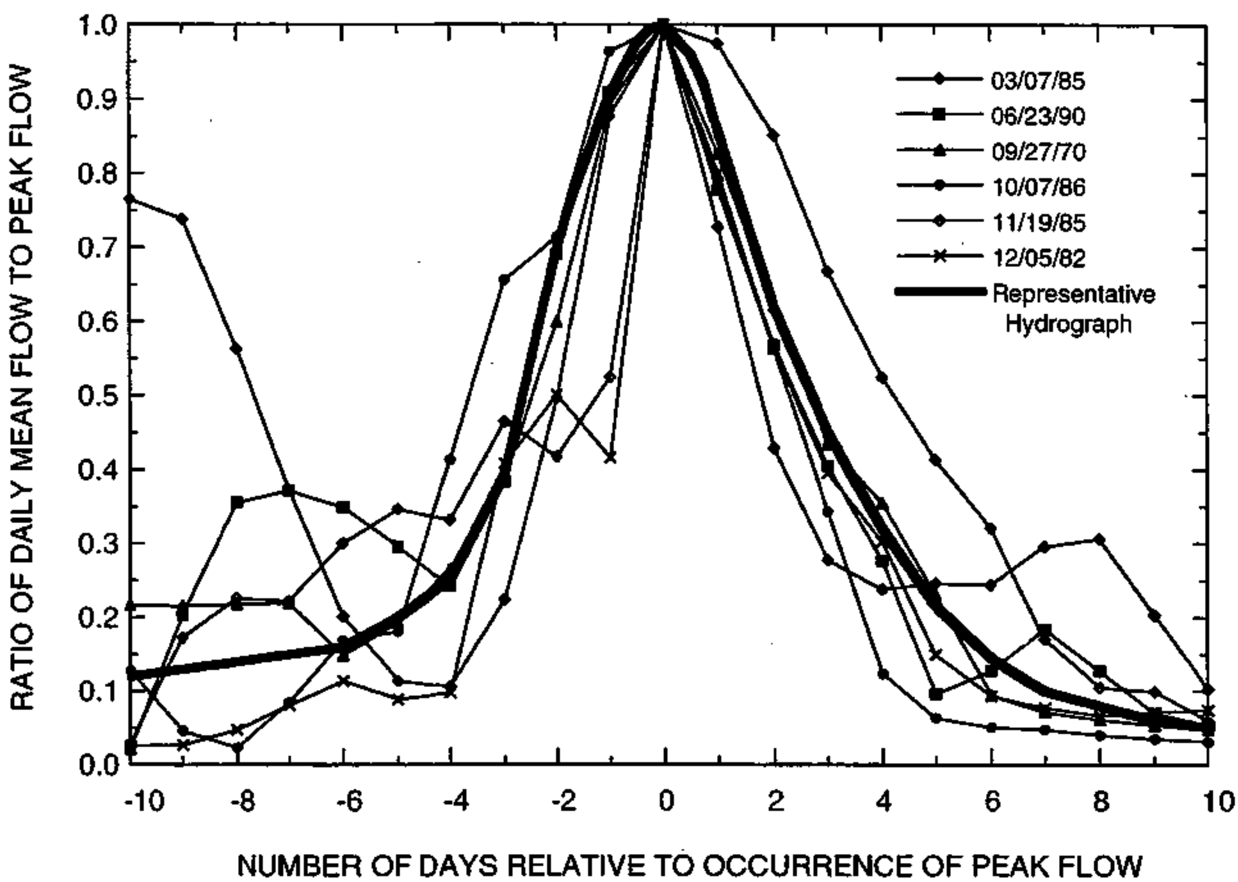


Figure 27. Normalization of the hydrographs of the top six floods, recorded at the La Moine River gage at Ripley, by their respective peak flows and selection of a representative discharge hydrograph.

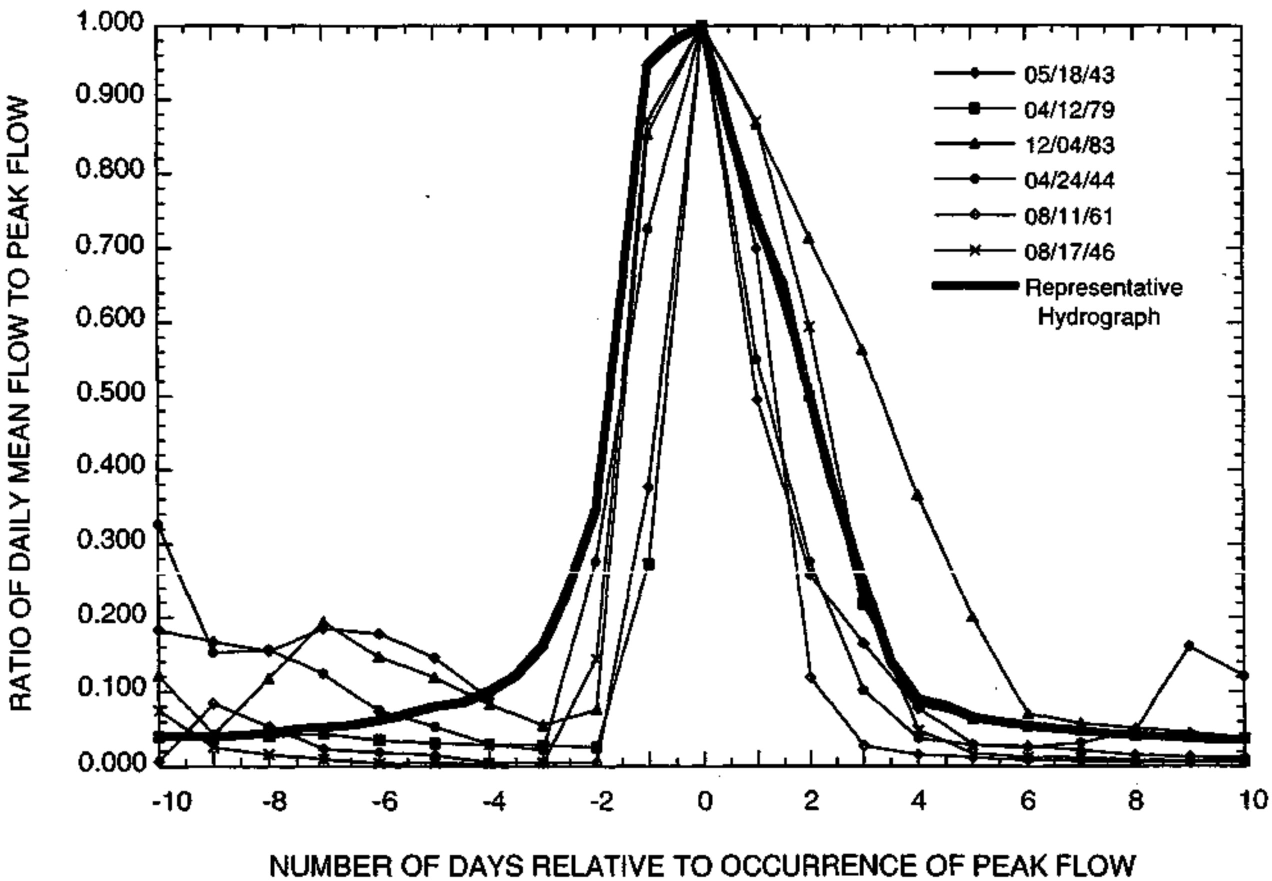


Figure 28. Normalization of the hydrographs of the top six floods, recorded at the Macoupin Creek gage near Kane, by their respective peak flows and selection of a representative discharge hydrograph

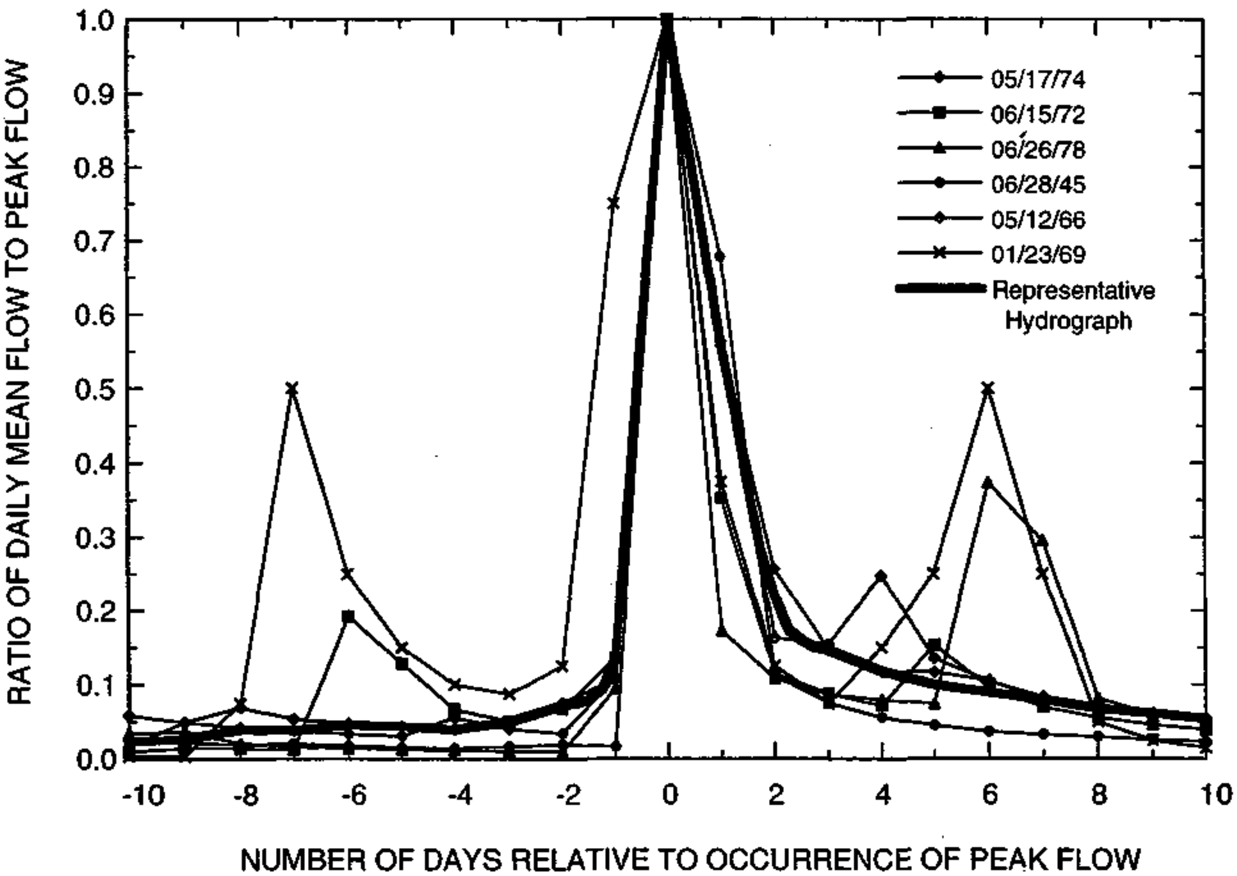


Figure 29. Normalization of the hydrographs of the top six floods, recorded at the Big Bureau Creek gage at Princeton, by their respective peak flows and selection of a representative discharge hydrograph

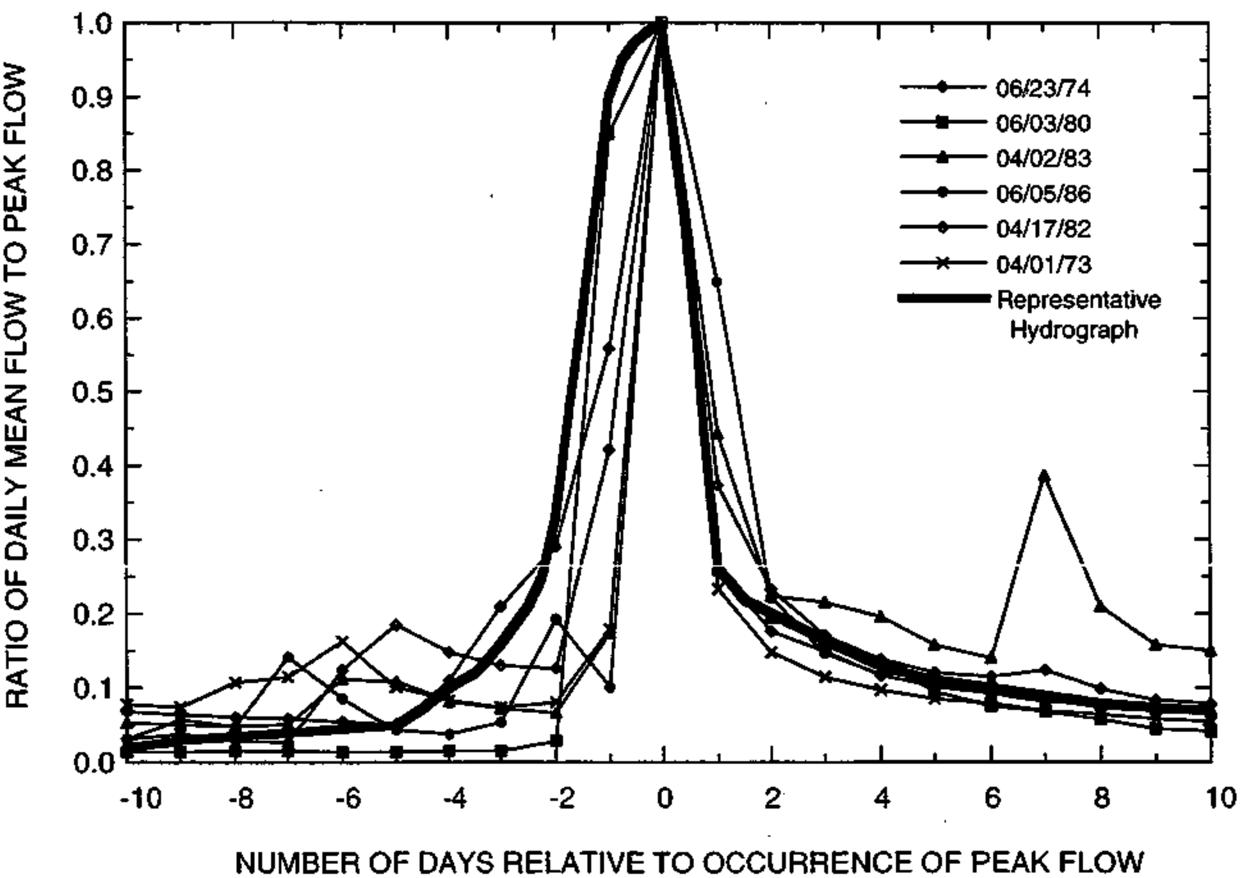


Figure 30. Normalization of the hydrographs of the top six floods, recorded at the Big Creek gage near Bryant, by their respective peak flows and selection of a representative discharge hydrograph

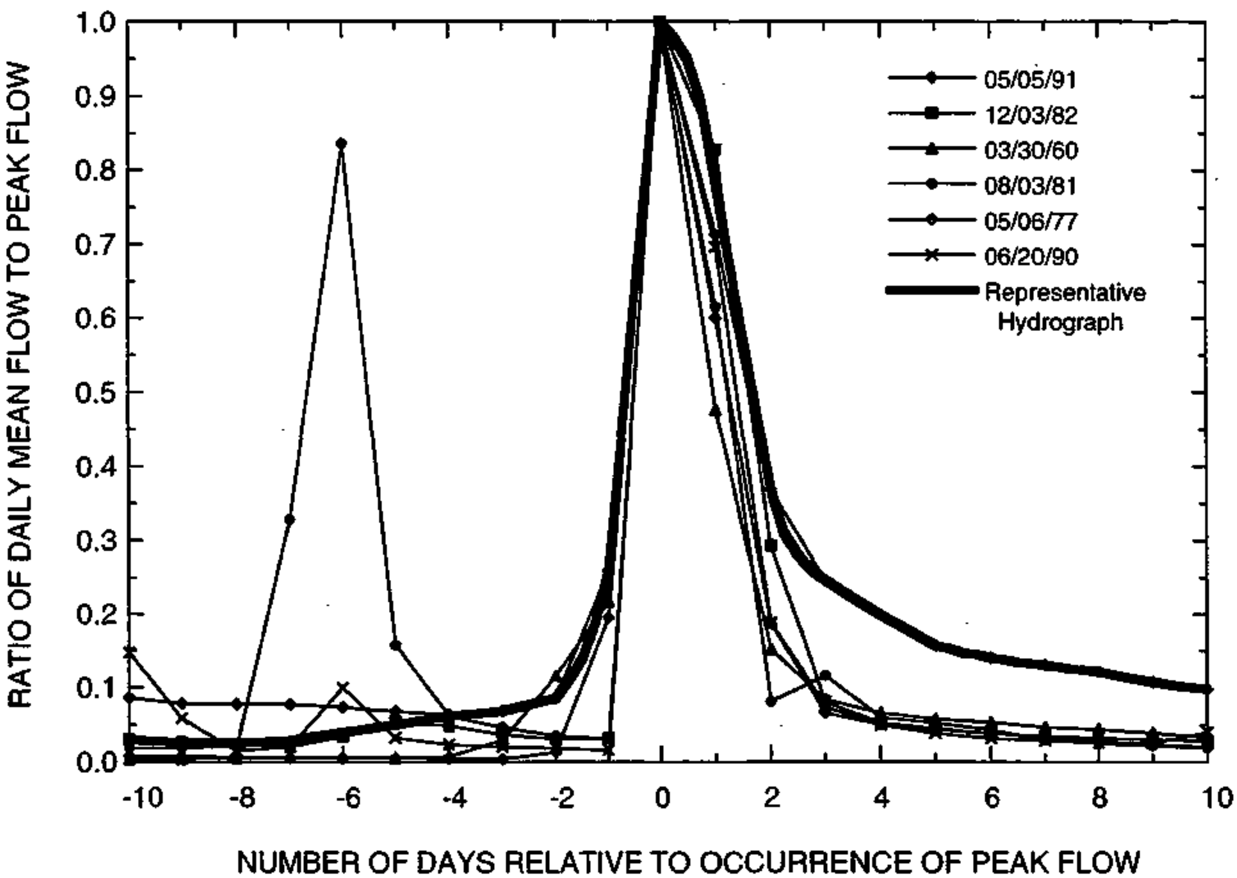


Figure 31. Normalization of the hydrographs of the top six floods, recorded at the Spring Creek gage at Springfield, by their respective peak flows and selection of a representative discharge hydrograph.

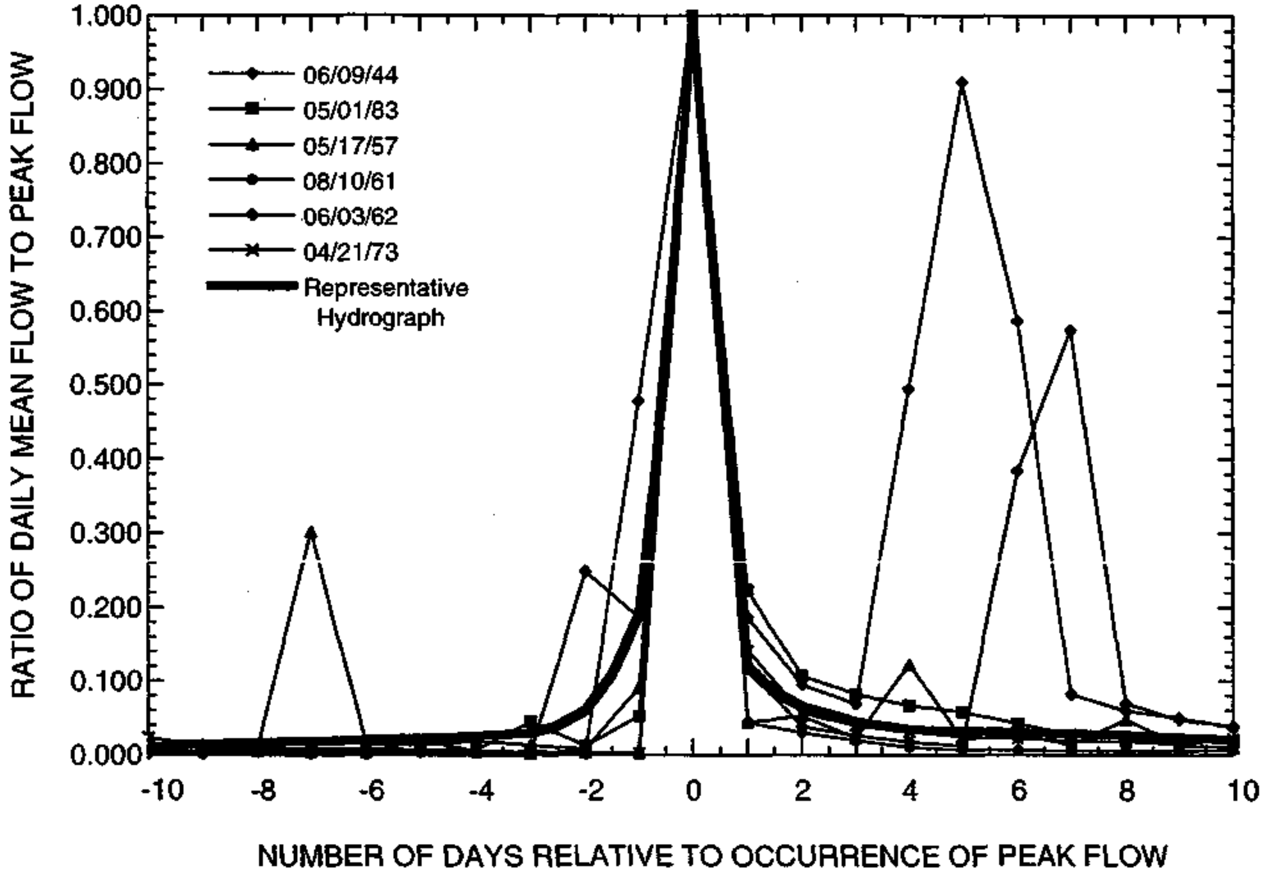


Figure 32. Normalization of the hydrographs of the top six floods, recorded at the Hadley Creek at Kinderhook gage, by their respective peak flows and selection of a representative discharge hydrograph

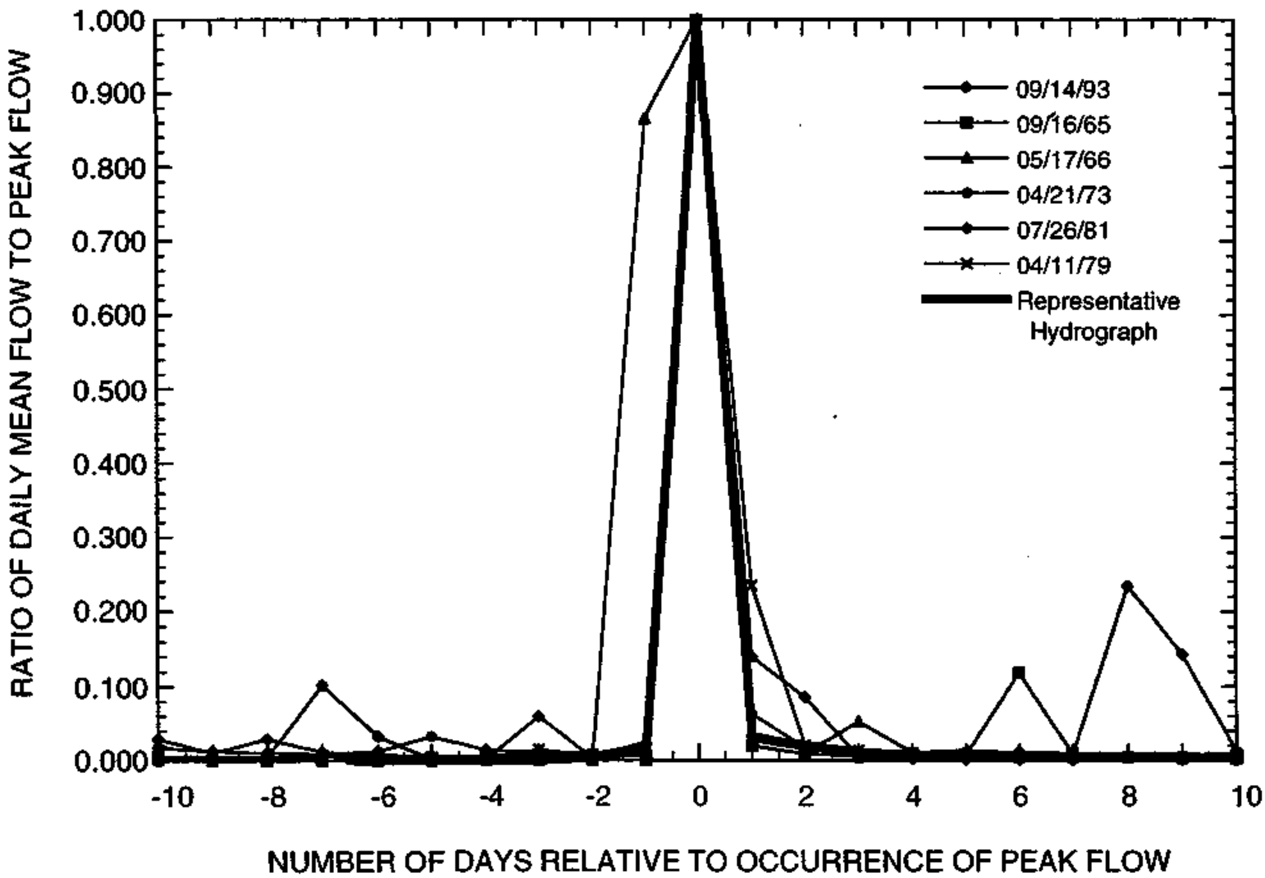


Figure 33. Normalization of the hydrographs of the top six floods, recorded at the Bay Creek at Pittsfield gage, by their respective peak flows and selection of a representative discharge hydrograph.

Table 10. Days Flood Peak at Meredosia Lags behind Those at Other Stations

	Macoupin		*Meredosia*	*Sangamon*		*La Moine*		*Spoon*		*Mackinaw*		*Kingston Mines*	
Year	Q_p	*Lag*	Q_p	Q_p	*Lag*	Q_p	*Lag*	Q_p	*Lag*	Q_p	*Lag*	Q_p	*Lag*
1943	40,000	-8	123,000	123,000	7	14,500	6	12,900	6	18,200	8	83,100	4
1985	19,400	-	122,000	26,500	5	23,000	3	29,200	4	18,400	-	78,800	4
1982	26,700	-8	112,000	68,700	7	21,000	6	21,000	7	51,000	7	88,800	5
1979	27,800	-7	111,000	55,900	4	3,120	7	12,600	6	23,400	-	72,300	4
1974	12,800	-	110,000	42,900	4	10,000	3	36,400	5	7,910	-	71,900	5
Representative Value of Lag		(-7) to (-8)			5-6		5-6		5-6		7-8		4

Note: Data have been taken from Table 25 of Singh (1996).
Q_p = Observed flood peak, cfs
Estimated lag for Peoria L&D is 5 days

type of analysis will be useful for predicting the highest floods that are feasible in the Illinois River.

The first set of simulation runs will involve the prediction of the 100-year peak WSE. Since the recurrence interval peak elevations for the stations have already been determined from the frequency analysis of stage data as shown in Table 11 (Singh, 1996), the objective is to compute water elevations that will match the 100-year elevations at the stations by adjusting the lateral inflows from the tributaries. First, the normalized WSE hydrographs at Peoria L&D, La Grange L&D and Grafton were determined (see Figures 34-36) and then multiplied by their respective peak WSE for the 100-year recurrence interval. These synthesized hydrographs will be used as upstream and downstream boundary conditions in the following simulations. Starting from Peoria, the next step is to match the 100-year peak WSE at Kingston Mines, the next station downstream of Peoria. This will be accomplished by starting with the 100-year discharge value for the gage on the Mackinaw River near Green Valley, adjusting this value gradually and using it after each adjustment to multiply the normalized hydrograph at the station until the computed peak WSE at Kingston Mines matched the 100-year water elevation. The goal in this effort is to compute a peak WSE that closely approximates the 100-year elevation. By so doing, it will be shown that the shape and duration of the computed hydrograph may not be a true representation of the actual 100-year hydrograph.

The final adjusted peak discharge for Green Valley will be checked with Table 9 to determine the recurrence interval of the Mackinaw River flow that contributed to the 100-year WSE in the Peoria-Kingston Mines reach of the Illinois River. The above procedure will be repeated for the remaining nine reaches between the gages downstream of Kingston Mines. As mentioned previously, the recurrence interval for the tributary flows will not necessarily be 100 years. The 10-, 25-, and 50-year water elevation profiles along the lower Illinois River will also be reproduced in a similar manner.

Effect of Changing Discharge and Variable Tributary Flows

The objective of these simulations will be to develop all feasible WSE profiles in the Illinois River from a combination of tributary and lateral inflows of various recurrence

Table 11. Peak Stages (cfs) for Gaging Stations in the Lower Illinois River at Various Recurrence Intervals

Gaging Station	*2-year*	*10-year*	*25-year*	*50-year*	*100-year*	*500-year*
Peoria L&DTW	447.02	452.30	454.43	455.92	457.28	460.21
Kingston Mines	445.68	450.82	453.02	454.55	456.00	459.19
Havana	442.49	447.79	450.11	451.68	453.14	456.26
Beardstown	439.08	445.51	447.91	449.52	451.02	454.21
La Grange L&D TW	437.62	443.73	446.20	447.90	449.48	452.92
Meredosia	436.72	442.92	445.38	447.08	448.59	451.89
Valley City	434.85	441.22	443.84	445.61	447.29	450.88
Florence	433.73	440.19	442.85	444.66	446.35	449.97
Pearl	431.18	437.44	440.15	442.07	443.91	448.01
Hardin	427.58	434.55	437.49	439.53	441.47	445.91
Grafton	425.23	432.52	435.81	438.09	440.24	444.86

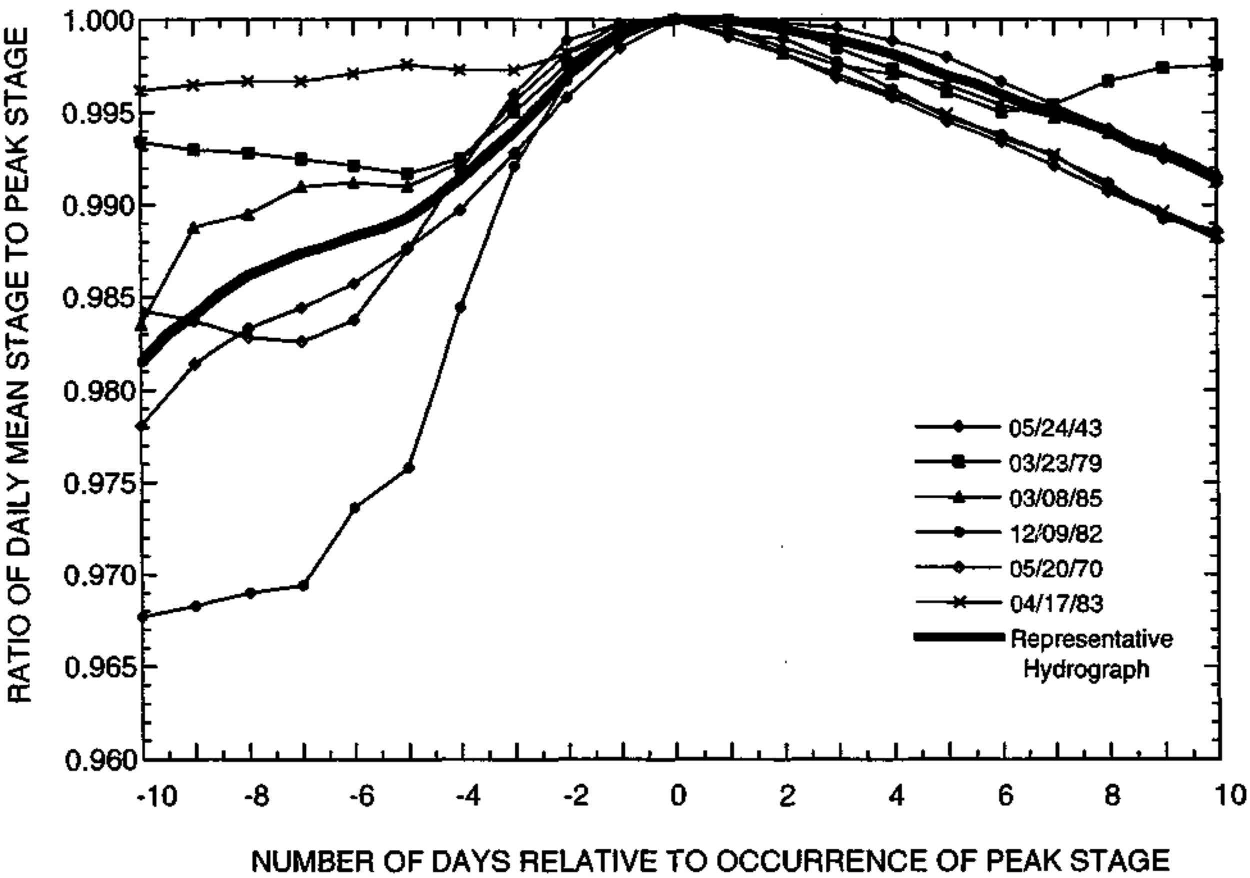


Figure 34. Normalization of the stage hydrographs of the top six floods, recorded at the Illinois River gage at Peoria Lock and Dam Tailwater, by their respective peak stages and selectionof a representative stage hydrograph

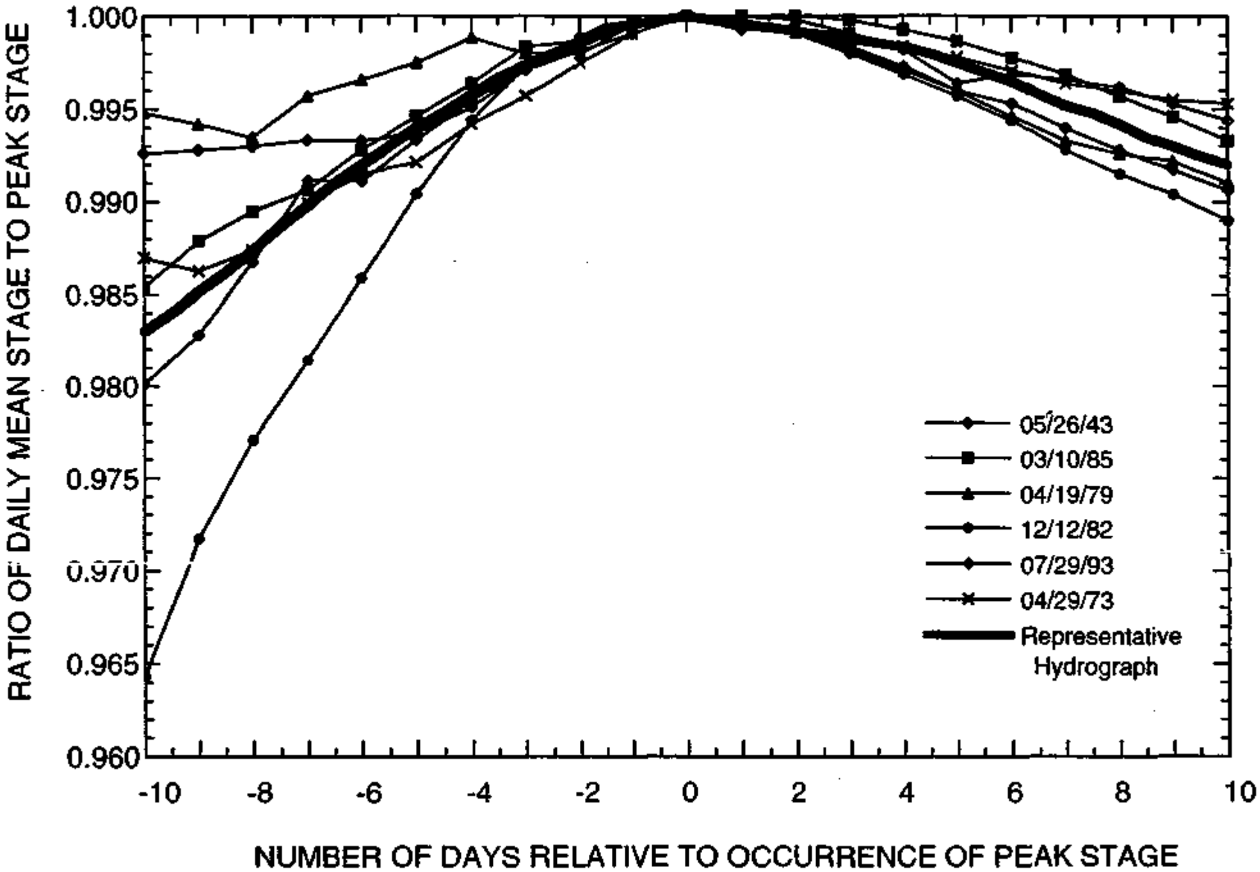


Figure 35. Normalization of the stage hydrographs of the top six floods, recorded at the Illinois River gage at La Grange Lock and Dam Tailwater, by their respective peak stages and selection of a representative stage hydrograph

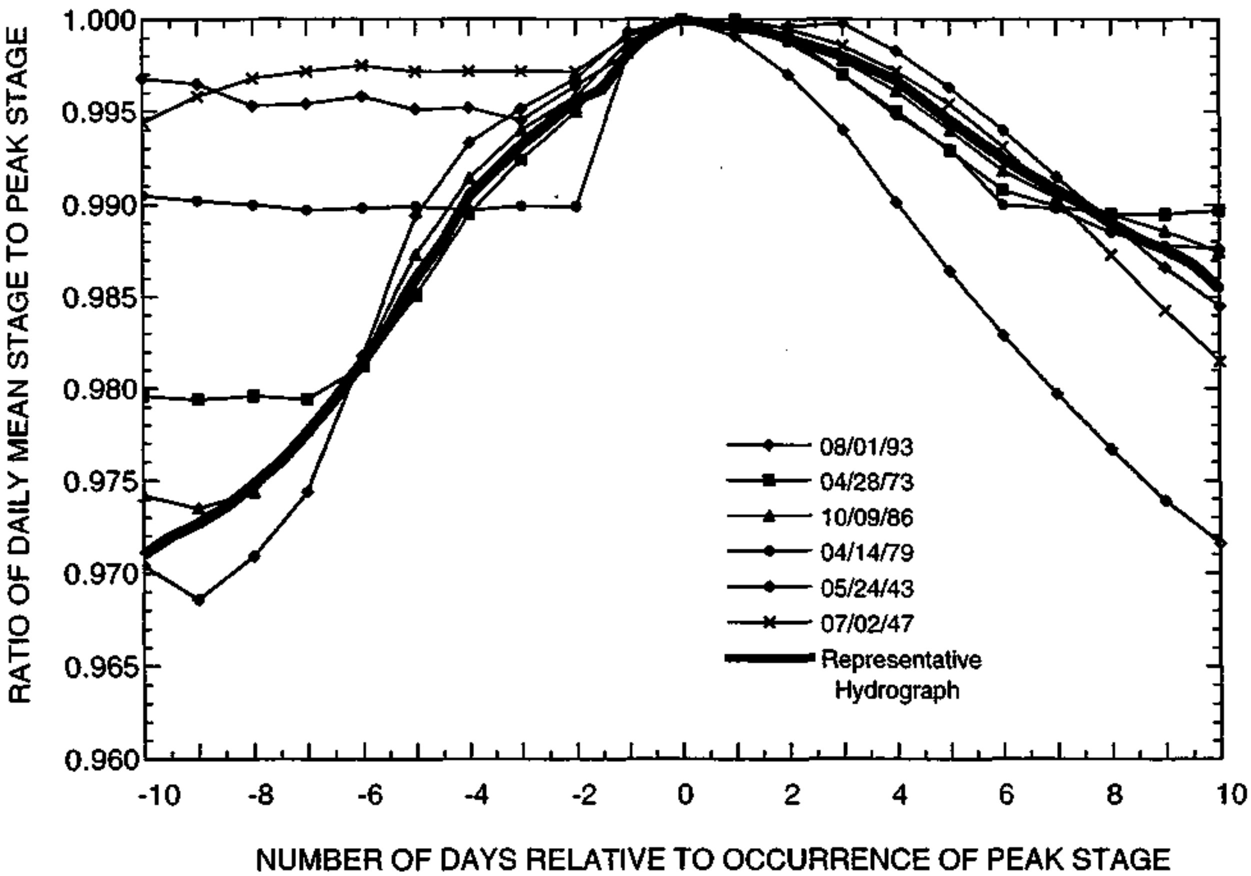


Figure 36. Normalization of the stage hydrographs of the top six floods, recorded at the Mississippi River gage at Grafton which is near the confluence of the Illinois River, by their respective peak stages and selection of a representative stage hydrograph

intervals. The analysis will also include the use of various time lags in such a way that the arrival of peak flow at the mouth of some of the tributaries coincide with the peak flood on the Illinois River. This flood coincidence analysis will be very useful in defining the feasibility of floods of higher magnitudes than has been observed to date and the probability of their occurrences. The recurrence probability of these higher floods will be extrapolated by plotting the return frequency floods on the same graph as the computed floods.

Examination of the flood stage profiles will help to determine the peak WSEs generated by the 10-, 25-, 50-, and 100-year flood stages at Kingston Mines and Meredosia.

Effect of Levee Flood Storage on Flood Peaks

The study of the reduction of peak flood stages due to flood storage in selected levees will be carried out mainly in the Alton Pool since the levees in this section of the Illinois River have been subjected to more frequent overtopping than the levees in the La Grange Pool because of the backwater effects from the Mississippi River. The simulations will examine existing conditions as well as other feasible situations.

Existing conditions will be examined in three ways. One series of simulation runs will be based on the assumption of a wall at the crown of the levees so that no levee overtopping will occur. Another set of runs will allow levee overtopping with provision for possibility of levee failures. In addition, the much often discussed situation of no levees on the Illinois River will be simulated by changing the cross-sectional elevations in the data input file. These simulations will be run for 10-, 25, 50-, and 100-year flows developed from the analyses in the previous section.

By selecting individual and combinations of levees in the Alton and La Grange Pools and creating openings 500-1000 feet wide and 4-6 feet deep along the crown of some of the levees, simulation runs for 10-, 25-, 50-, and 100-year flow will be performed. The results of these simulations in comparison to the previous simulation runs will be used to determine the reductions in peak flood that can be attained due to the conversion of some of the levees to temporary storage areas and to evaluate the duration of holding of water in these storage basins.

SUMMARY

The unsteady flow model, UNET, has been tested and validated for the lower Illinois River reach between Peoria L&D and Grafton. The model was applied to simulate the May 1979 and March 1985 floods, which are ranked, respectively, fourth and second at Meredosia, and sixth and third at Kingston Mines. The computed water surface elevations were compared with the recorded events at eight stage-gaging stations between Peoria L&D and Grafton. The computed WSE hydrographs for the 1979 flood event fit the observed data generally within 0.5 feet for the stations in the La Grange Pool. However, the model errors were generally over one foot at the stations in the Alton Pool. The deterioration in the accuracy of the one-reach model (Peoria L&D to Grafton) for the Alton Pool led to the development of a two-reach model (Peoria L&D to La Grange L&D and La Grange L&D to Grafton). The computed two-reach WSE for the 1979 flood fit the observed WSEs more closely than the one-reach model.

The 1985 flood was also simulated with the one-reach and the two-reach models, but the one-reach model results were better than the two-reach model results. However, the differences between observed and computed peak WSE for the two models were less than 0.5 feet. The two-reach model calibrations for the 1979 flood provided better approximation of the observed peaks than the 1985 simulations. However, the stage hydrographs were better predicted for the 1985 flood than for the 1979 flood. Because of the mixed results, the roughness coefficients adopted for subsequent analyses were obtained by averaging the roughness coefficients for the 1979 and 1985 calibrations.

Further testing of the UNET model for the lower Illinois River included the simulation of the December 1982, June 1974, April 1973, and July 1993 flood events. The two-reach model WSE profiles for these floods fit the observed data much better than the one-reach model profiles. The two-reach model will, therefore, be adopted for all future simulations.

The predicted WSE for the 1982 flood event were all within 0.5 feet of the observed data at all the gaging stations. The predicted hydrographs lagged the observed hydrographs by one to two days at the stations downstream of Meredosia. At Hardin, the

crest of the predicted WSE hydrograph was broader than the observed hydrograph. The differences in the computed and observed WSE hydrographs may be due to errors caused by the variation in the timing of the flood flow from Macoupin Creek.

The computed WSE hydrographs for the June 1974 flood event closely matched the observed hydrographs at all the stations, but there were lags of up to one day in the peak WSE hydrographs from Valley City to Hardin. The model underestimated the observed peak WSE for the 1973 flood by less than 0.25 feet at the stations. The peaks of the WSE hydrographs were generally in phase with the observed hydrographs, but there were differences of up to 0.75 feet in the peak WSE between Florence and Hardin.

The 1993 peak discharges at Meredosia and Kingston Mines were estimated to have return periods of approximately three to five years. The low return periods at these stations implied that the high stages in the Illinois River in 1993 were not due to a major flood on the river. Instead, the prolonged and unprecedented flooding on the Mississippi River, which produced the highest historical stage at Grafton, caused backwater effects that extended upstream on the Illinois River. The backwater effects were shown to affect stages at Havana (R.M. 119.6), but it did not seem to affect stages at Kingston Mines (R.M. 145.6). The prediction of the 1993 flood event was shown to be accurate within 0.5 feet of the observed peak WSE.

Work in Progress

The model simulations, for the analysis of flood-stage reduction due to levee storage options, required the development of typical discharge hydrographs for each of the tributary stations and stage hydrographs at Peoria L&D, La Grange L&D, and Grafton. They also required the estimation of appropriate time lags for the tributary station hydrographs. This information will be used in the development of the 100-year stage profile along the Illinois River and for the generation of all feasible flood stages and their probability of occurrence.

The ordinates of the six top floods at the gages on the five major tributaries (Mackinaw, Spoon, Sangamon, and La Moine Rivers, and Macoupin Creek) and at the five stations representing the ungaged tributaries (Big Bureau, Big, Spring, Hadley, and

Bay Creeks) were normalized with the peak discharges of corresponding hydrographs. Representative normalized flood hydrographs were constructed for each station and then converted to actual flood hydrographs by multiplying the ordinates by the corresponding 10-, 25-, 50-, and 100-year discharges for each station. These synthetic hydrographs will be used in the simulation of WSE profiles for the various return periods. Appropriate time lags for the synthetic hydrographs have been determined from the timing of the flood peaks for the 1943, 1974, 1979, 1982, and 1985 flood events. The lags for the gaged tributaries were estimated as approximately between six and seven days relative to Meredosia.

The first series of simulations that is currently underway includes the prediction of the 100-year WSE profiles for the La Grange and Alton Pools. Starting from Peoria, the tributary flows will be adjusted until the peak WSE at the Illinois River gages closely match the 100-year flood stages derived from the frequency analyses of the gage records. The next task will be to develop of all feasible WSE profiles obtained by combining tributary and lateral inflows of various recurrence intervals and adjusting the time lags of the discharge hydrographs so that the timing of some of the tributary floods coincides with those on the Illinois River.

The final stage of the simulations will be the prediction of peak stage reductions for levee storage options mainly in the La Grange Pool and some in the Alton Pool. The model simulations will include scenarios of 1) no levee failure by assuming a vertical wall along the crowns of the levees to determine maximum flood stages, 2) the flood stages along the Illinois River if the levees had not been constructed, and 3) the restricted flood stages for safety of levees with some of them converted to the levee storage option.

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