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Management Strategies for Flood Protection in the Lower Illinois River

Phase I: Development of the Lower Illinois River - Pool 26 UNET Model

by
David T. Soong and Yanqing Lian

Prepared for the
Office of Water Resoures
Illinois Department of Natural Resources

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Illinois State Water Survey
Watershed Science Section
Champaign, Illinois

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Contents

Page

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Abstract

One of the main concerns was the ability to specify proper stage hydrographs at the downstream boundary of the Lower Illinois River for hydraulic design and analysis. We found that a unique stage-discharge rating relationship does not exist at the lower boundary of the Lower Illinois River at Grafton because the backwater effects from the Upper Mississippi River. Management options and results for managed storage and emergency activities need to be analyzed under more comprehensive design of flooding conditions. To improve the capability of UNET for modeling backwater effects for the Lower Illinois River, an extended model including Pool 26 of the Upper Mississippi River was developed. The downstream stations of the model are at the tail of Lock and Dam 25 and the Mel Price Lock and Dam Pool, where stage readings are available. The model was calibrated with a 1979 flood and verified with a 1983 flood. Discharge and stage frequency analysis have also been performed for stations at Troy on Cuivre River, Lock and Dam 25 tail, Lock and Dam 26 pool, and Lock and Dam at Mel Price on the Mississippi River.

Introduction

Morphologic features of the Lower Illinois River from Peoria to Grafton include wide river valleys, steep bluffs, and flat channel slopes. Most of the floodplains have been reclaimed and included in organized Levee Drainage Districts (LDDs) for agricultural purposes (Illinois State Planning Commission, 1940). Construction of the LDDs was also mostly completed between 1879 and 1916 (Thompson, 1989), and levee heights in practically all districts had conformed to a grade line, at least 4 feet above the 1926 flood level, approved by the federal government (Illinois State Planning Commission, 1940). Figure 1 shows the locations of currently active LDDs along the Lower Illinois River. Reported levee heights could afford protection at a level approximating 20- to 50-year return intervals (Singh, 1996; USACOE 1994).

With existing buildings on the floodplain along the main channel, these levees apparently could not meet the floodplain encroachment regulations later specified by the Federal Emergency Management Agency or FEMA (1987). That agency defined the Regulatory Floodway as that portion of the floodplain that must be reserved from encroachment in order to pass a 100-year flood without increasing the water-surface elevation more than 1 foot, providing hazardous velocities are not produced. Studies have shown that the alignments of levees could affect the flood elevations at various magnitudes (Hall, 1991); there are real concerns about the actual levels of protection the levee system can provide.

Changes in estimated flood heights also affect the level of protection a levee system can provide. There are indications of increased flood heights since 1970 due to a trend of increasing precipitation in the upper half of the Illinois River basin (Singh and Ramamurthy, 1990). The

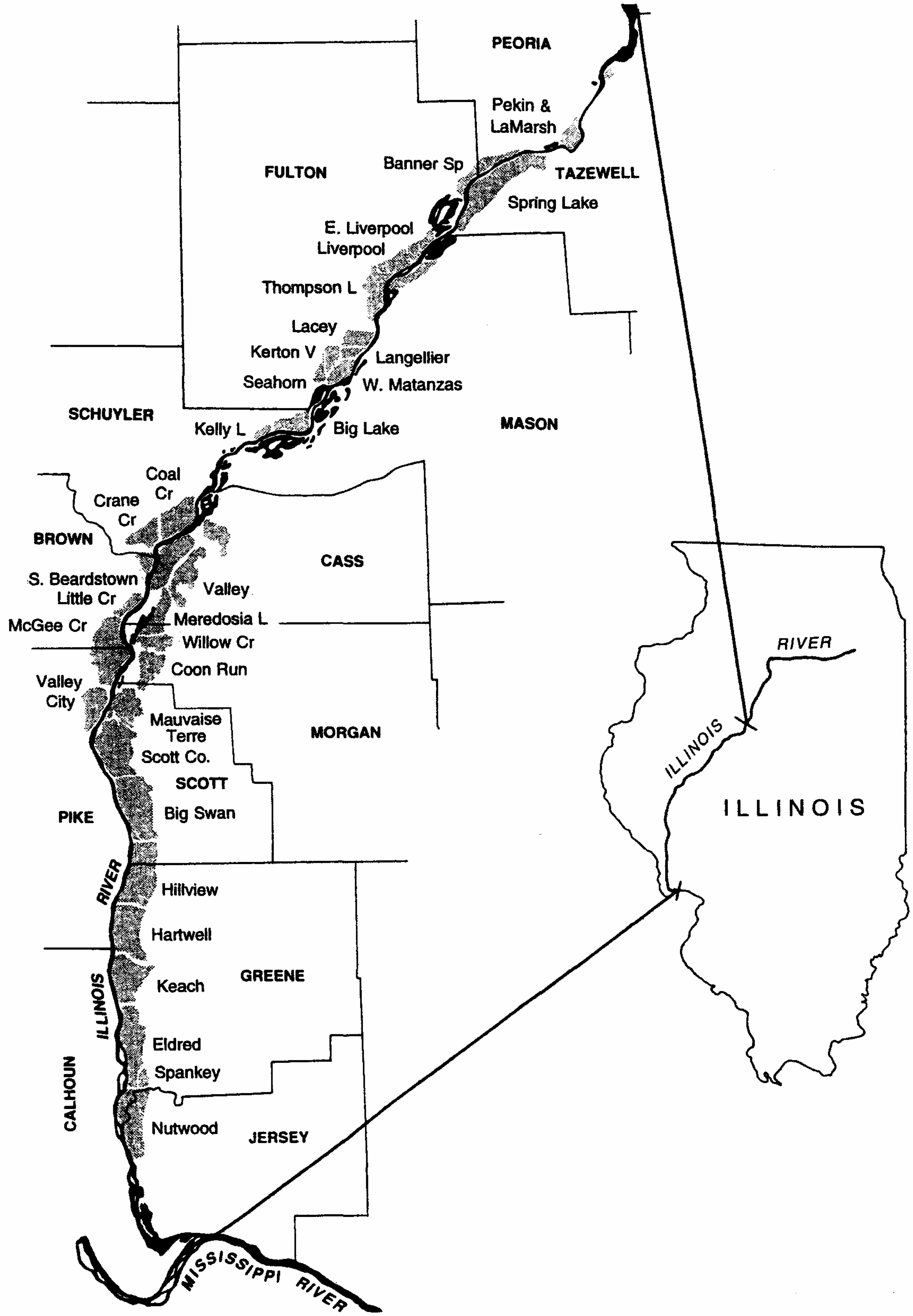

Figure 1. Levee and drainage districts in the LaGrange and Alton Pools of the Lower Illinois River (From Akanbi and Singh, 1997)

higher flood peaks have further increased the risk of overtopped or breached levees in many instances. During a 1985 flood, for example, the stage exceeded the top elevations of levees at Globe, Coal Creek, Lost Creek, and South Beardstown LDDs between Kingston Mines and Meredosia; during a 1993 flood, levees at Nutwood, Eldred, Hillview, Hartwell, Spankey, and elsewhere were overtopped or breached (USACOE, 1994). Floods have produced and have the potential to produce devastating damages and trauma, and they can impede economic growth and development of a region. There is an urgent need to develop management plans and flood protection strategies for the Lower Illinois River.

Difficulties in predicting flood heights render flood protection decisions problematical. Levee failures involve geotechnical, hydrologic, and human factors (Figure 2). Only the hydrologic aspects of flood protection and management will be discussed here. Table 1 can be used to illustrate the variability of peak stages from several selected floods. It can be seen that the famous 1993 Flood was a downstream flood, while others could be considered upstream floods in the study reach. Also worth noting are the responses of stages in the reach along Meredosia (River Mile or RM 71.3) to different floods.

Hydraulic analysis and numerical modeling can assist greatly in flood protection and management planning. Advances in numerical modeling have enabled us to simulate flood

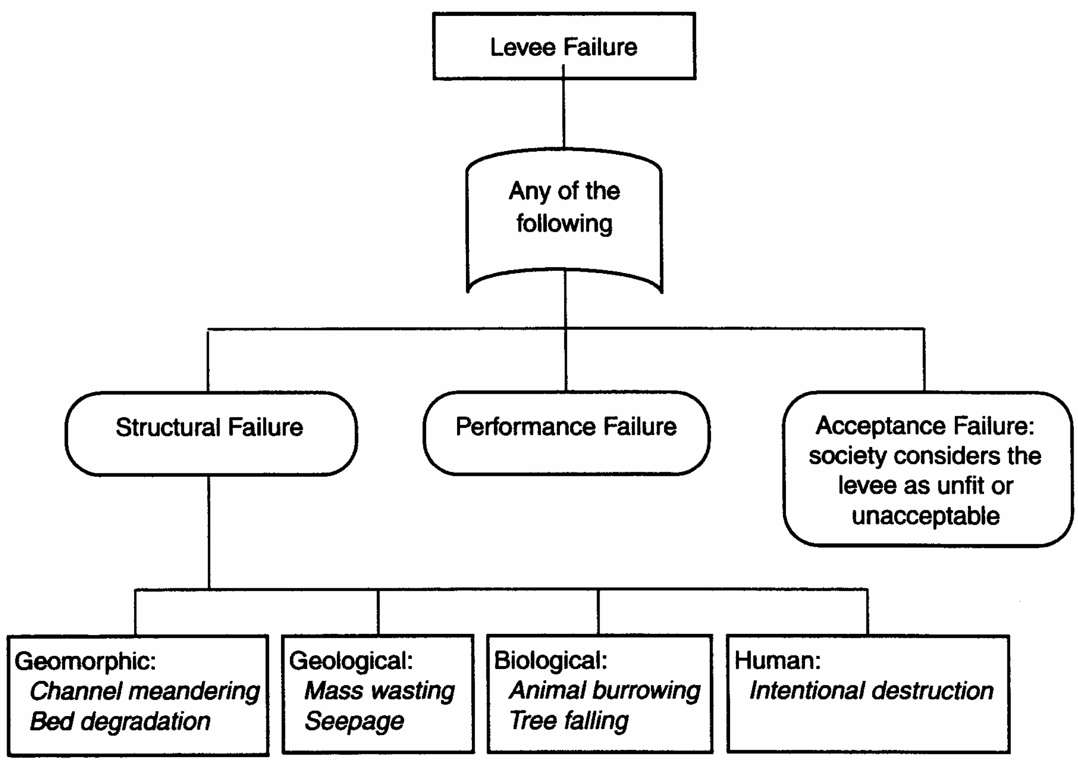

Figure 2. Types of failure in a levee system

Table 1. Observed Flood Stages in Lower Illinois River, feet above msl

River Mile	*1995*	*1993*	*1985*	*1982*	*1979*	*1943*
157.9	453.40	450.95	455.60	454.80	455.70	456.00
152.9				453.60	454.90	455.20
145.4	452.34	449.53	453.70	452.40	453.20	454.00
136.8	451.66	448.36	451.96	451.50	452.00	452.50
128.6				450.40	451.20	452.00
119.6	450.66	447.90	450.84	449.50	450.00	451.60
97.3				448.00	448.60	450.40
88.1	449.19	446.56	448.40	447.20	448.00	449.60
80.2	447.48	445.95	446.60	445.60	446.40	447.20
71.3	446.36	444.96	445.62	444.40	445.28	446.70
61.6	444.39	444.39	443.60	442.30	443.20	444.90
56		443.60	442.10	441.00	441.90	442.80
43.2		442.75	438.55	437.45	439.70	439.10
31.5						436.60
21.6		442.40	434.00	433.75	436.50	434.80
0		441.94	430.47	431.50	433.17	432.70

propagation along channels, in addition to determining the water surface profiles for a given set of discharges and stages at channel boundaries. One can use the peaks and durations of the calculated flood hydrographs to evaluate potential for overtopping and/or breaching at areas of concern in the study reach. By properly designing the management scenarios, one can therefore investigate and evaluate these practices prior to actual implementation. A model also can be run with recent data retrieved from the Internet for use in a real-time prediction mode. Recommendations and options resulting from this study not only will serve as the basis for management practices but also be of tremendous value when decisions need to be made during flooding situations. To maximize the use of results from this investigation, we envision a computer-simulated screen-in-screen presentation of up-to-date water surface profile and management options as a product of the project. This tool needs to be flexible and easy to use. Nonetheless, the main focuses of this project are to conduct rigorous hydraulic analyses on the existing levee-channel system and to recommend management and flood protection options for the Lower Illinois River.

Background

Stage reductions were noted following levee breaches on the Upper Mississippi River during the 1993 Flood (Figure 3). The breached LDDs effectively provided temporary storage and hence stage reductions until the LDDs were filled. These stage drops were sufficient to sustain certain flood-fighting actions. The LDD system provides not only for the levees to withhold floods in the channel but also opportunities for mitigating the flood hazards. A series of projects sponsored by the Office of Water Resources of the Illinois Department of Natural

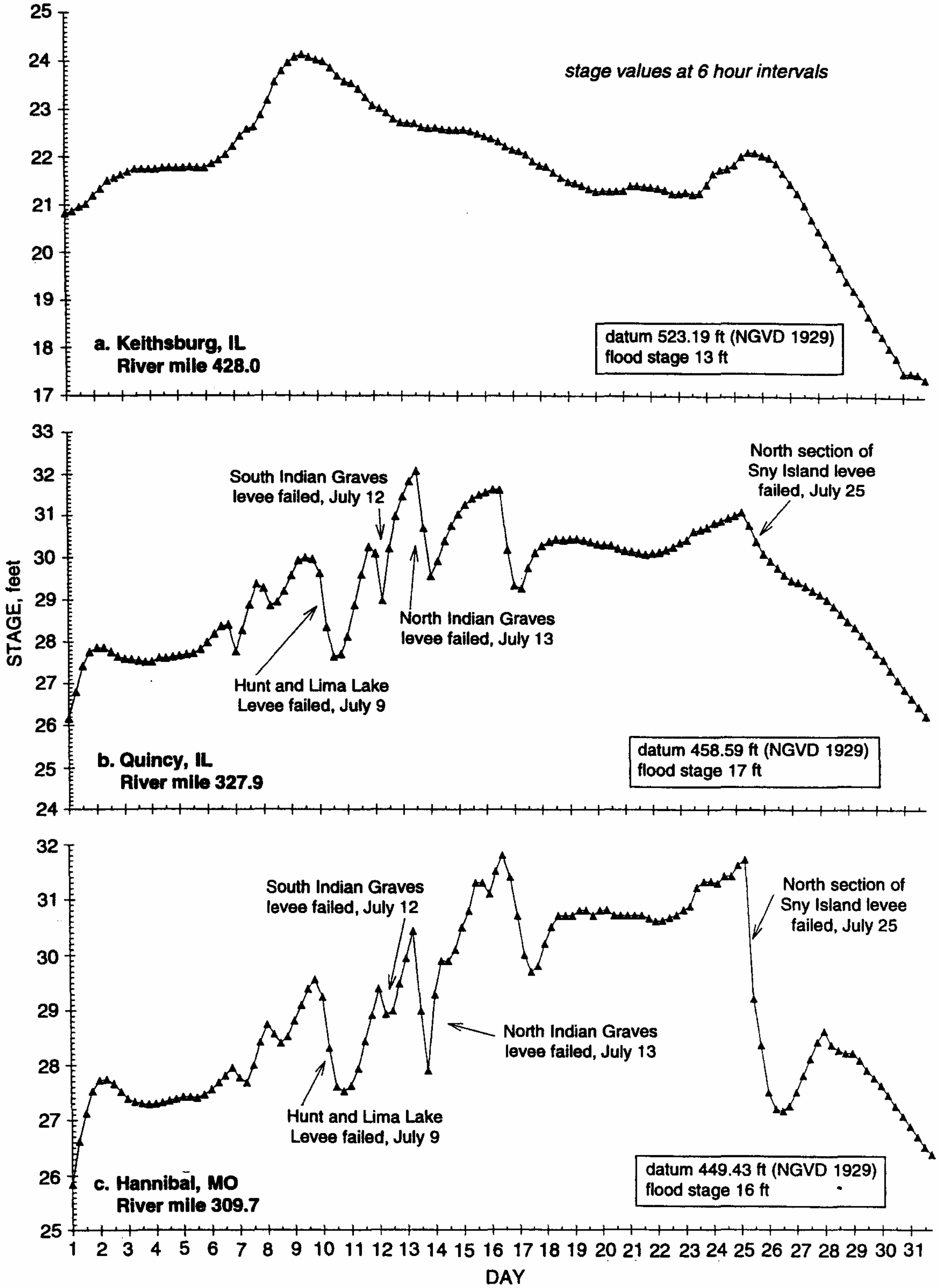

Figure 3. Mississippi River at Keithsburg, Quincy, and Hannibal, Missouri, July 1993 (From Bhowmik et al., 1995)

Resources (IDNR) has been conducted by the Illinois State Water Survey (ISWS) to examine the managed flood storage option for selected LDDs along the Lower Illinois River. Studies have analyzed stage and flood frequencies for the Mississippi backwater effects (Singh, 1996), validated a UNET unsteady flow model (Barkau, 1995) for the La Grange and Alton Pools of the Lower Illinois River (Akanbi and Singh, 1997), and evaluated the managed flood storage options using the model (Akanbi et al., 1999). The results showed that reductions in excess of 1 foot in peak stages near Meredosia could be achieved with a combination of LDDs converted to flood storage areas. If the Lacey, Langellier, W. Matanzas & Kerton Valley, and McGee Creek LDDs were converted to managed storage areas, the additional area protected against a 100-year flood could reach 65,262 acres upstream of RM 43.2. On the other hand, levees downstream of this section would have to be raised by 1 - 3 feet to protect them against a 100-year flood on the Upper Mississippi River.

Managing levees for flood protection in the Lower Illinois River is a broad plan that involves concerns other than using LDDs for storage alone. During the flood events of the 1990s local emergency activities such as using sandbags and flood fences or boards to raise levee heights were common. Besides the ability to predict locations with the potential for overtopping or experiencing prolonged high stages, it is also beneficial to know if flood protection efforts will be effective for the type of flood that is occurring and the risks to the working crews. Looking at the system as a whole, the decision makers also need to know how locally added levee heights would affect flood stages at areas up- or downstream, and what combinations of emergency activities and managed storage would be effective and permissible. A detailed hydraulic analysis could essentially answer these questions, and the success of the previous 1999 study is the impetus for the current investigation. That study laid the foundation for a detailed investigation on the flood protection and management of the lower Illinois River. In addition, the 1999 study examined managed flood storage options for one set of boundary conditions – a 100-year flood from upstream and the 100-year stage at Grafton. Management options need to be developed with more comprehensive design of flooding conditions.

The Scientific Literature

This section reviews literature, reports, and data specifically related to floods, flood protection, and management plans especially for the Lower Illinois River. The scope of this section will be expanded gradually in future reports. The present purpose is to report the knowledge gained from a physical model study of the Mississippi River, including a portion of the Lower Illinois, Missouri, and Ohio Rivers.

The Water Experiment Station (WES) of the U.S. Army Corps of Engineers conducted the study on the Mississippi River Model, which was part of the Mississippi Basin Model or MBM (Foster, 1977). The MBM extended on the main stem of the Mississippi River from Hannibal, Missouri, to Baton Rouge, Louisiana. Its uppermost station on the Illinois River was Meredosia, Illinois; the uppermost station on the Missouri River was Hermann, Missouri; and the uppermost station on the Ohio River was Galconda, Illinois. The scale of the fixed bed model was 1:2000 (horizontal) and 1:100 (vertical).

The MBM was used to develop steady-state water surface profiles on the Mississippi and Illinois Rivers to assist in a) updating rating curves, b) re-establishing design grades for authorized levee projects, and c) developing data for economic benefit and flood insurance studies. These profiles also were used to ascertain which river flow-line (the Mississippi, Illinois, or Missouri) would produce the highest stages at points along the Mississippi and Illinois Rivers for particular frequencies. The latter part is of particular interest to the hydraulic design portion of this investigation. Additionally, the report also documented water surface elevations for all scenarios in the channel at all model gaging stations and other points necessary to give a detailed profile at approximately 1-mile intervals along the various levee units. The test procedures used flows of 5- to 500-year return periods, the agricultural and urban design floods on one river, and flows on the remaining two rivers to maintain the correct flow frequency on the tested river. Note that the Mississippi River urban design flood with a crest discharge of 1,300,000 cubic feet per second (cfs) at St. Louis, Missouri (equivalent to the flood of record in 1844) is the flood for which the existing St. Louis levee and floodwall are designed. The Illinois River agricultural design flood is a flood equal to the May 1943 Illinois River flood coincident with a 50-year flood on the Mississippi River. The frequency of this flood varies somewhat along through the 80-mile reach of the Illinois River, but it is approximately equivalent to the 100-year event on the Illinois River.

In conclusion, the experimental tests indicated no levee on the Illinois River was crevassed when:

- The modified agriculture design flow was tested on the Illinois River with the authorized levee installed. The stage at Grafton was held to the elevation obtained for testing of the 50-year Mississippi River flow-line with authorized levees.
- Eight combinations of flows representing Illinois River flow-lines from 5- to 500-year frequency (i.e., 5-, 10-, 25-, 50-, 100-, 200-, agriculture design, and 500-year) were tested in the Hannibal to St. Louis reach with St. Louis District authorized levees installed. The downstream control at Chain of Rocks used stages obtained with flows for the Mississippi River flow-line tests from 5- to 50-year frequency.

Although several levee-breaching floods occurred after the study, the report can be used for studying impacts from the Missouri River, for model comparisons, for comparison of the measured steady-state water surface profile from the MBM and from the frequency analysis, and other studies in addition to examining combinations of floods.

Overall Goals and Objectives

The primary emphasis of the proposed work is to perform rigorous hydraulic analyses on the Lower Illinois River levee-channel system and to develop recommendations and management options for flood protection. Additionally, if the input data are retrieved from the Internet, the model can run in a pseudo-real-time mode – delayed only by the time lag involved in data reporting and retrieval from the source. At the end of the research our results will be integrated into the pseudo-real-time mode to formulate a tool for management and flood-fighting concerns. This hydrologic tool for flood protection and management planning will include five major elements:

1. An appropriate wave-routing program for the Lower Illinois River.
2. Pseudo-real-time access to the current data source and forecasts.
3. Dynamic interfaces to the comprehensive databases for hydrologic and geometry data as well as analyzed management options.
4. On screen presentation of up-to-date flood status and management options as well as predicted results.
5. Windows-driven instructions to run computer models.

The whole project is expected to be completed in 3.5 years. The objective of this report is to document progress during in the first phase (half year), which included constructing the Lower Illinois River - Pool 26 UNET model, updating Data Storage System (DSS) files, and performing stage and discharge frequency analyses for the new stations. Future reports will describe the hydraulic analysis conducted on the existing levee-channel configuration, managed storage options for more comprehensive boundary conditions, and flood-fighting issues.

Acknowledgments

The Office of Water Resources (OWR), Illinois Department of Natural Resources (IDWR), provided external funding for this investigation. Mr. Gary Clark and Mr. Robert Dalton, both of OWR of IDNR, provided information and many helpful suggestions during the project. Gary Clark also serves as the project manger. Personnel of the Water Control Section of the U.S. Army Corps of Engineers - St. Louis District, including Mr. Dennis Stephen and Ms. Tori Goessling, provided the geometry file of the recent UNET model and stage/discharge data for relevant stations in the study reach. The authors also thank staff of the Illinois State Water Survey for their efforts and assistance in this project. Ms. Karla Andrew helped with data retrieval and organization for the Data Storage System, Mr. Mike Myers conducted the literature search and compared the cross-sectional geometry, Ms. Eva Kingston edited the report, and Ms. Linda Hascall prepared final graphs for the report. A sincere appreciation is also extended to students from the University of Illinois at Urbana-Champaign who worked very hard on this project. Mr. Miguel Retrepo checked the computer programs for the frequency analysis and rewrote a FORTRAN version of the main program. Ms. Kristin Jaburek conducted numerous tests for each station with the frequency analysis program and prepared the output.

The Lower Illinois River - Pool 26 UNET Model

Description

Modeling the hydraulics of a low-gradient river like the Lower Illinois River requires special attention to backwater effects and in-channel/off-channel storage. The UNET program (Barkau, 1995) is appropriate in such an environment because it solves the full dynamic wave equation that includes variations in convective and local accelerations as well as static pressure terms. It also is formulated to account for main-channel and floodplain hydraulics and has adopted computational routings for evaluating the levee system and other human-made structures. In applying the program to field models, one initial condition and two boundary conditions must be specified. However, one of the main concerns about the 1999 Lower Illinois River UNET model was the ability to specify proper downstream boundary conditions in the analysis. With the backwater effects from the Upper Mississippi River at the confluence, a unique stage-discharge rating relationship does not exist at this site.

The backwater is a hydraulic phenomenon at the confluence of the Lower Illinois River and Pool 26 of the Mississippi River. This investigation resolves it by including Pool 26 in the model. Figure 4 shows a schematic diagram of the connections of the Lower Illinois River - Pool 26 UNET model. It contains 7 reaches with reaches 4, 5, 6, and 7 describing Pool 26. Data for the Pool 26 portion were obtained from a portion of the Mississippi River UNET model that was provided by the U.S. Army Corps of Engineers, St. Louis District. Table 2 illustrates the comparisons of the two components of the new model.

The original Lower Illinois River UNET model used cross-sectional data measured in the 1971-1979 period (mostly measured in 1978); however, the Pool 26 UNET model used cross sections measured in 1995. Some discrepancies have been noted and will be discussed later in this report. The Lower Illinois River - Pool 26 UNET model also uses a newer version of the UNET model (V3.2; HEC, 1997) that has levee breaching functions.

Application of a numerical program to field conditions is accomplished by establishing representative cross sections and connecting cross sections with reach lengths. The model becomes representative after calibrating the parameters in the governing equations and assigning proper boundary conditions.

Boundary Conditions

Boundary conditions specified for the Lower Illinois River - Pool 26 UNET model could be the stage or discharge hydrographs. On the main stems, stage hydrographs, either actual or determined from frequency analysis, are used at the tail of Peoria Lock & Dam (L&D), at the tail of L&D 25, and at Mel Price L&D pool. Discharge inputs are specified for the many tributaries along the Illinois River and Pool 26 of the Mississippi River. Since some of the tributaries have measured data and some do not, specifications of boundary conditions for tributaries are varied. Tributaries other than the Sangamon River were explicitly specified as point inputs to the Illinois River, while the Sangamon River was modeled as a tributary with the uppermost station at Oakford. Similarly the Cuivre River was modeled as a tributary of Pool 26, and its input was defined at Troy. For ungaged tributaries along the Illinois River,

Akanbi and Singh (1997) scaled the selected gage with the fraction of the area between the two watersheds to determine the point input. For much smaller tributaries they used uniform lateral inflow, which was estimated by scaling the discharge records of the gaging stations on a nearby watershed with the fraction of "unbalanced" drainage area to the area of hydrologically similar watershed. In order to run the model correctly, an initial stage is specified and the model starts with a steady-state run first. The initial stage can be estimated from past records or from the established rating curve.

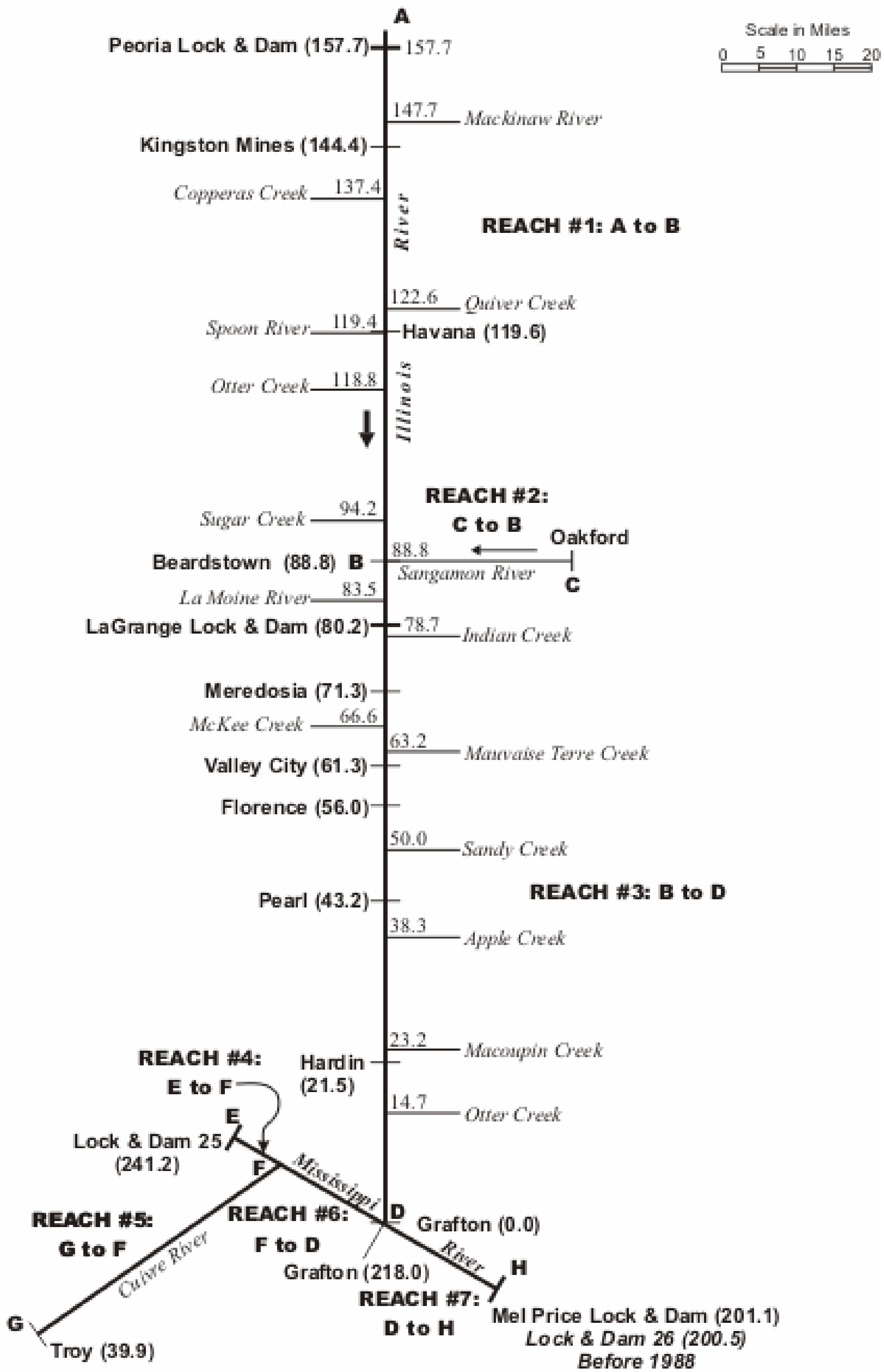

Figure 4. A schematic of the seven reaches of the Lower Illinois River-Pool 26 UNET model

Table 2. Descriptions of the Lower Illinois River and Pool 26 Models

Variable	*Lower Illinois River model*	*Pool 26 model*
River Mile	0-157.7	200.61-241.2
Structures	La Grange L&D and bridges	1 bridge
Boundary stations	Peoria L&D tail water, stage at Grafton	L&D 25 tail water, Mel Price L&D pool
# of cross sections	378	131
Years measured	Mostly in 1978	1995
LDDs modeled	26	None
Major tributaries modeled	Sangamon River (25.7 miles, 33 cross sections)	Cuivre River (39.9 miles, 10 cross sections)

In UNET modeling users can specify internal boundary conditions at connection points between reaches. By specifying internal boundary conditions, users can obtain better control of the computed results. However, this is not always possible if no data are available. No internal boundary conditions are specified in the new model. It is not feasible to specify two elevations at the La Grange L&D (both pool and tail water) in the model, and this study will test the matches at Grafton by checking the computation error at the flow junction. Once the model is calibrated, this internal boundary condition can be added to improve model performance.

Data Storage System Update

The DSS information for the river being modeled with UNET has been updated to include all the stations needed in the Lower Illinois River - Pool 26 UNET model for their available stage or discharge data. Current efforts involve the data verification and organization of the data in the DSS database. Table 3 shows the stations, type of data, and duration for data prepared to be used in the DSS.

The downloaded data will require checking for consistency and missing data. Further investigation to improve the estimation of ungaged tributaries is also needed.

Model Calibration and Verification

The calibration procedures involve adjusting the ratios of conveyance change or the discharge-conveyance factors for a specific reach in the boundary condition file until the computed stages match the measured data for selected events. In this sense, the events selected for calibration and verification need to be representative of the river reach. With the establishment of a comprehensive database for available gaging stations, the model can be run for any given time period available in the database, and it will therefore present a great flexibility for selecting events for calibration and verification purposes. However, the large database also requires careful scrutiny for errors. In addition, estimating discharges for ungaged tributaries using existing data series always presents challenges in modeling.

Table 3. An Overview of Data Prepared for the DSS

Stations	*ID*	*Source*	*Type*	*Duration*
Illinois River at Peoria	5560000		Flow	1903-1906 1910-1938
Peoria L&D Tail		Rock Island COE	Stage	1988-1999
Bay Creek at Pittsfield	5512500	USGS	Stage	1993-1998
Bay Creek at Pittsfield	5512500	USGS	Flow	1939-1999
Big Bureau Creek at Princeton	5556500	USGS	Stage	1993-1998
Big Bureau Creek at Princeton	5556500	USGS	Flow	1936-1999
Illinois River at Beardstown	5584000	USGS	Flow	1920-1938
Illinois River at Florence		COE, St Louis	Stage	1930-1938; 1942-2000
Illinois River at Hardin	5587060	USGS	Stage	1987-1998
Illinois River at Hardin	5587060	COE, St Louis	Stage	1878-1880; 1932-2000
Illinois River at Havana	5570500	USGS	Flow	1921-1927; 1985-1989
Illinois River at Kingston Mines	5568500	USGS	Stage	1993-1998
Illinois River at Kingston Mines	5568500	USGS	Flow	1939-1999
Illinois River at Pearl		COE, St Louis	Stage	1878-1881; 1885-1938; 1942-2000
Illinois River at Valley City	5586100	USGS	Flow	1938-1999
Illinois River at Valley City	5586100	COE, St Louis	Stage	1883-1999
Illinois River near Copperas Creek		Rock Island COE	Stage	1988-1999
Illinois River near Havana	5570500	Rock Island COE	Stage	1988-1999
Illinois River near Kingston Mines	5568500	Rock Island COE	Stage	1988-1999
Illinois River near Meredosia	5585500	USGS	Stage	1988-1999
Illinois River near Meredosia	5585500	Rock Island COE	Flow	1989; 1991-1994
L&D 25, Tail water		COE, St Louis	Stage	1938-1999
L&D 25 Pool		COE, St Louis	Stage	1939-1995
L&D 26 Pool		COE, St Louis	Stage	1938-1995
L&D 26 Tail		COE, St Louis	Stage	1891-1990
L&D 26 Tail		COE, St Louis	Stage	1992-1995
Mel Price Pool		COE, St Louis	Stage	1990-1999

Table 3. An Overview of Data Prepared for the DSS (Concluded)

Stations	*ID*	*Source*	*Type*	*Duration*
Mel Price Tail water		COE, St Louis	Stage	1990-1995
La Moine River at Ripley	5585000	USGS	Stage	1993-1998
La Moine River at Ripley	5585000	USGS	Flow	1921-1999
Mackinaw River at Congerville	5567500	USGS	Stage	1993-1998
Mackinaw River at Congerville	5567500	USGS	Flow	1944-1999
Mackinaw River near Green Valley	5568000	USGS	Stage	1993-1998
Mackinaw River near Green Valley	5568000	USGS	Flow	1921-1956
Mackinaw River near Green Valley	5568000	USGS	Flow	1988-1999
Macoupin Creek near Kane	5587000	USGS	Stage	1993-1998
Macoupin Creek near Kane	5587000	USGS	Flow	1921-1999
Mississippi River at Alton	5587498	COE, St Louis	Stage	1990-2000
Mississippi River at Alton	5587498	USGS	Flow	1933-1987
Mississippi River at Dixon		COE, St Louis	Stage	1930-2000
Mississippi River at Grafton	5587450	COE, St Louis	Stage	1879-1904; 1929-1999
Mississippi River at Grafton	5587450	USGS	Flow	1933-1998
Mississippi River at Keokuk	5474500	USGS	Flow	1878-1999
Mississippi River at St. Louis	7010000	USGS	Flow	1933-1999
Missouri River at Hermann	6934500	USGS	Flow	1928-1998
Sangamon River near Oakford	5583000	USGS	Stage	1993-1999
Sangamon River near Oakford	5583000	Rock Island, COE	Stage	1988-1999
Sangamon River near Oakford	5583000	USGS	Flow	1909-1911; 1914-1919; 1921-1922; 1928-1933; 1939-1999
Spoon River at Seville	5570000	USGS	Stage	1993-1998
Spoon River at Seville	5570000	USGS	Flow	1914-1999
Spring Creek at Springfield	5577500	USGS	Stage	1993-1998
Spring Creek at Springfield	5577500	USGS	Flow	1948-1999
Troy on Cuivre River	5514500	USGS	Flow	1922-1998
Mississippi River at Grafton	5587450	COE, St. Louis	Flow	1929-1999

Lai et al. (1991) has shown that when different combinations of boundary conditions (e.g., stage - discharge instead of stage - stage) are used, the parameter calibration could lead to different values. Akanbi and Singh (1997) showed the results of calibration and verification using a two-reach model from Peoria L&D to La Grange L&D, and then from La Grange L&D to Grafton. The calibration results from the whole reach model were reportedly not as good as those from the two-reach model. The two-reach model has been calibrated with flood hydrographs of 1979 and 1985 and verified with 1973, 1974, 1982, and 1983 floods. The current approach is a one-reach modeling method; therefore the calibration and verification are more challenging. The relaxation of downstream boundary conditions has significantly changed the originally established model parameters. In an effort to continuously improve model performance, no internal boundary conditions are set to force better matches.

The current model is calibrated with the 1979 flood and verified with the 1983 flood event. Figures 5 - 17 show the calibrated and observed hydrographs at 13 stations on the Lower Illinois River and the Pool 26 of Mississippi River. The most upstream boundary at Peoria L&D was excluded because the match at boundary stations is mandatory. Figures 18 - 30 show the verified stage hydrographs for the same ten stations. Due to the limitations of mathematical formulation and perhaps, more importantly, the ability to specify ungaged watersheds at present, stations in the middle reach of the model experience more obvious discrepancies. The current strategy is to make sure the same parameter set can work for all flood events.

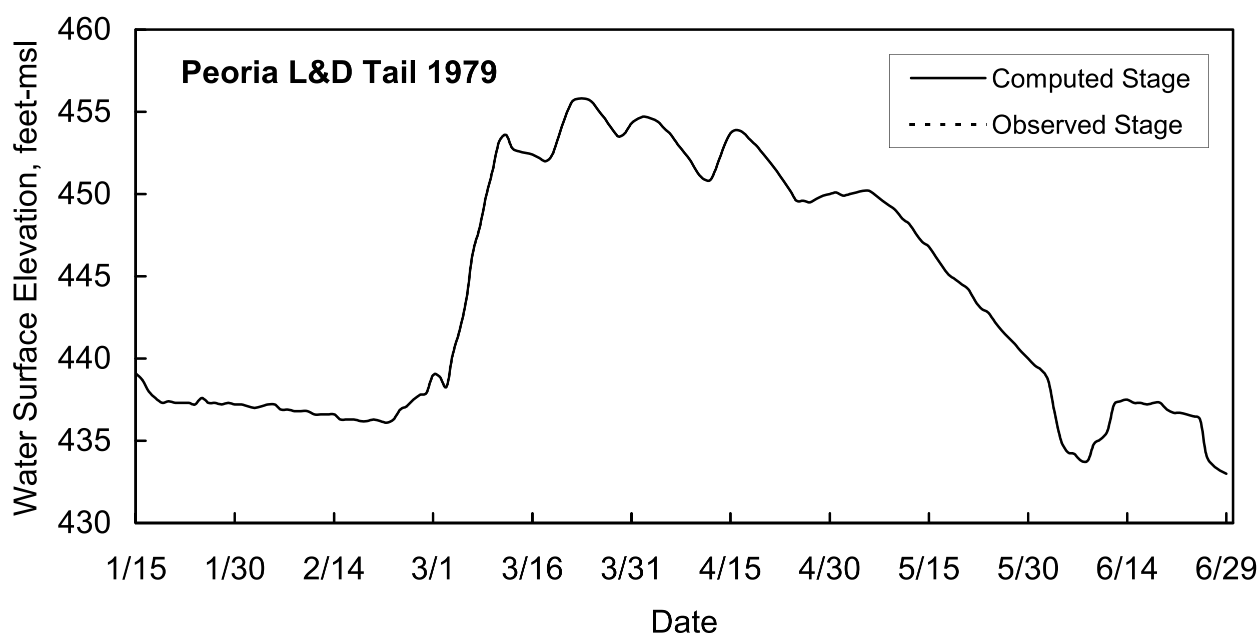

Figure 5. Computed and observed stage, Peoria Lock and Dam tail, 1979

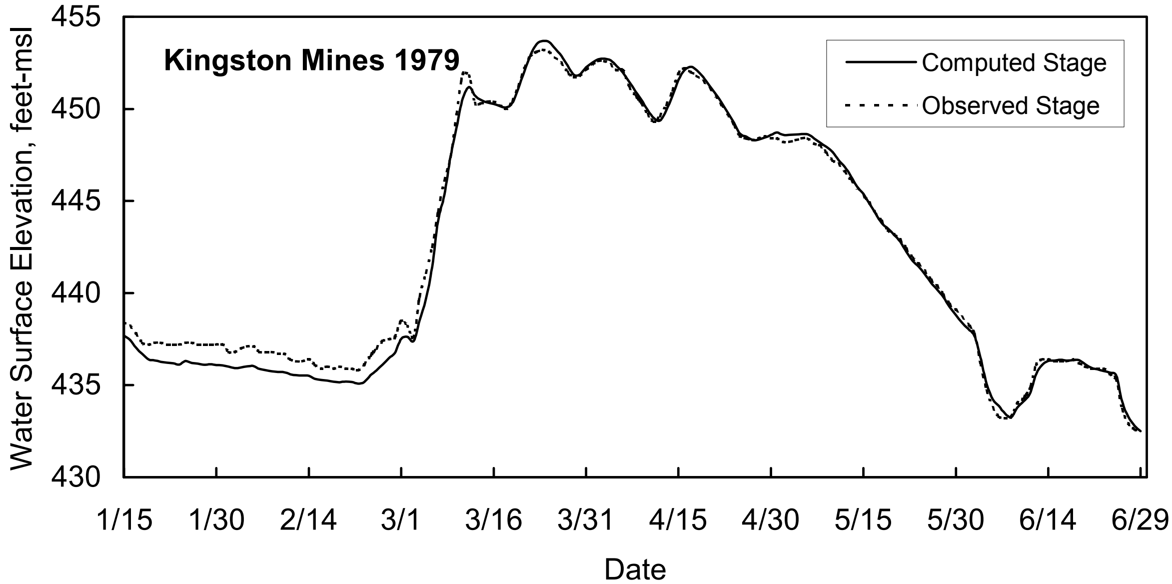

Figure 6. Computed and observed stage, Kingston Mines, 1979

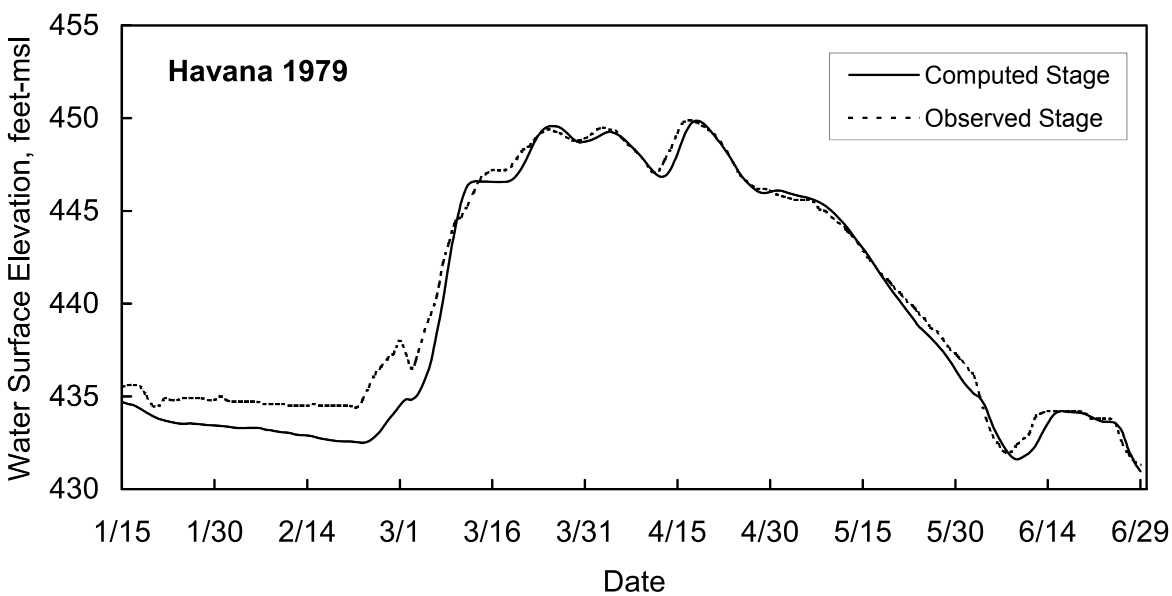

Figure 7. Computed and observed stage, Havana, 1979

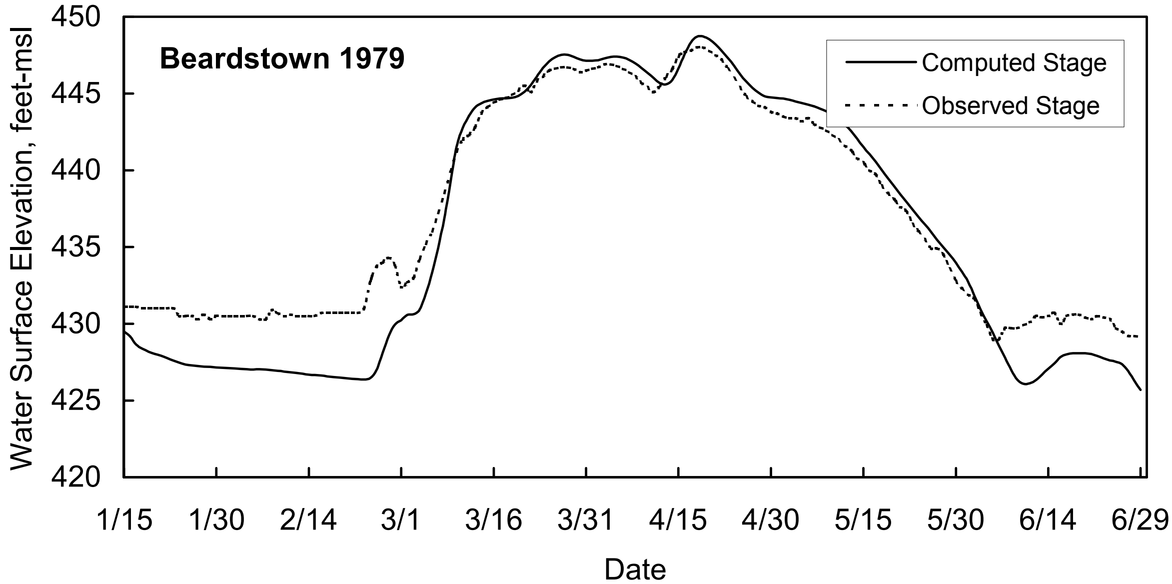

Figure 8. Computed and observed stage, Beardstown, 1979

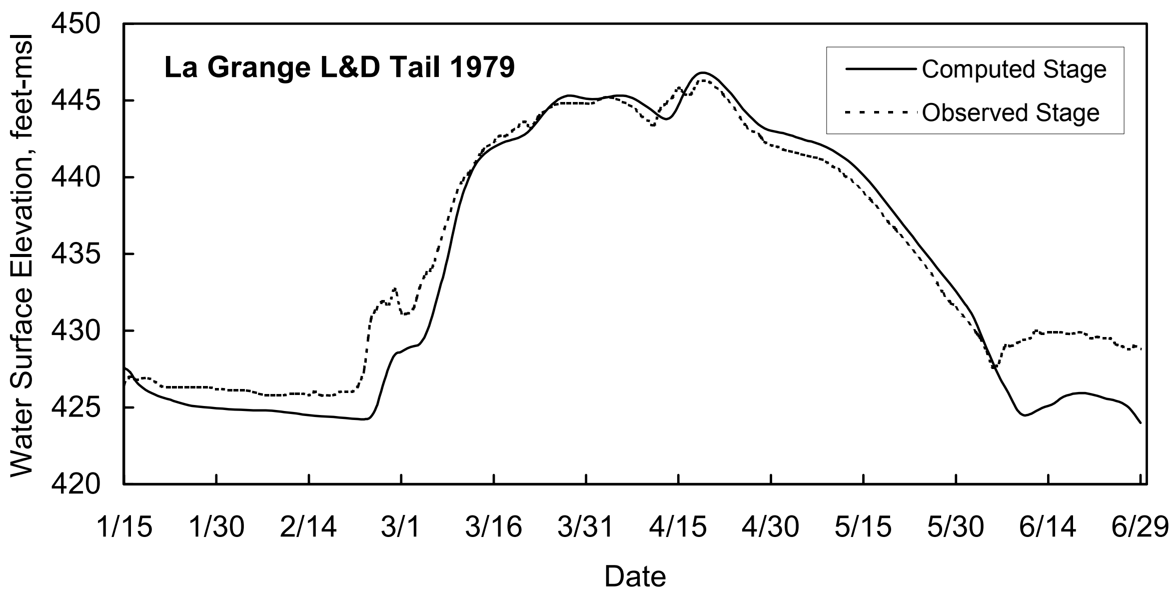

Figure 9. Computed and observed stage, La Grange Lock and Dam tail, 1979

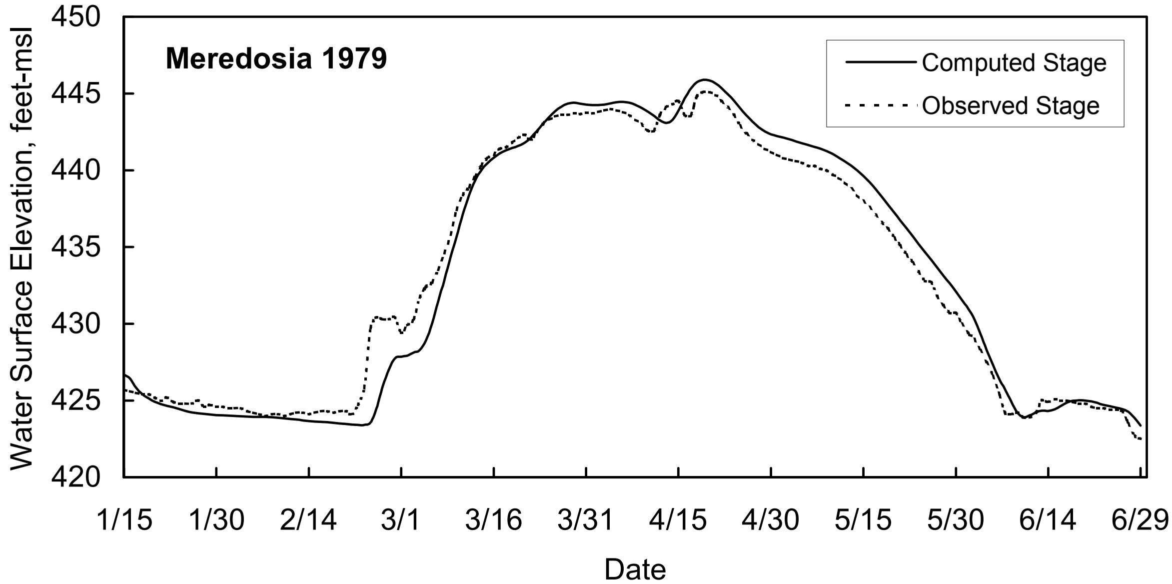

Figure 10. Computed and observed stage, Meredosia, 1979

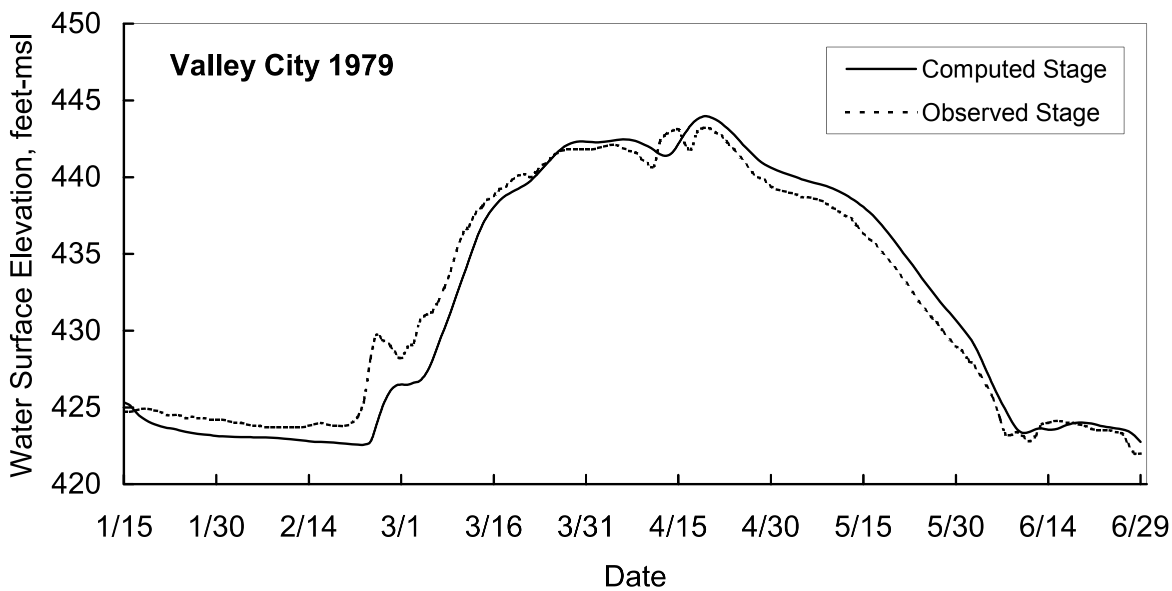

Figure 11. Computed and observed stage, Valley City, 1979

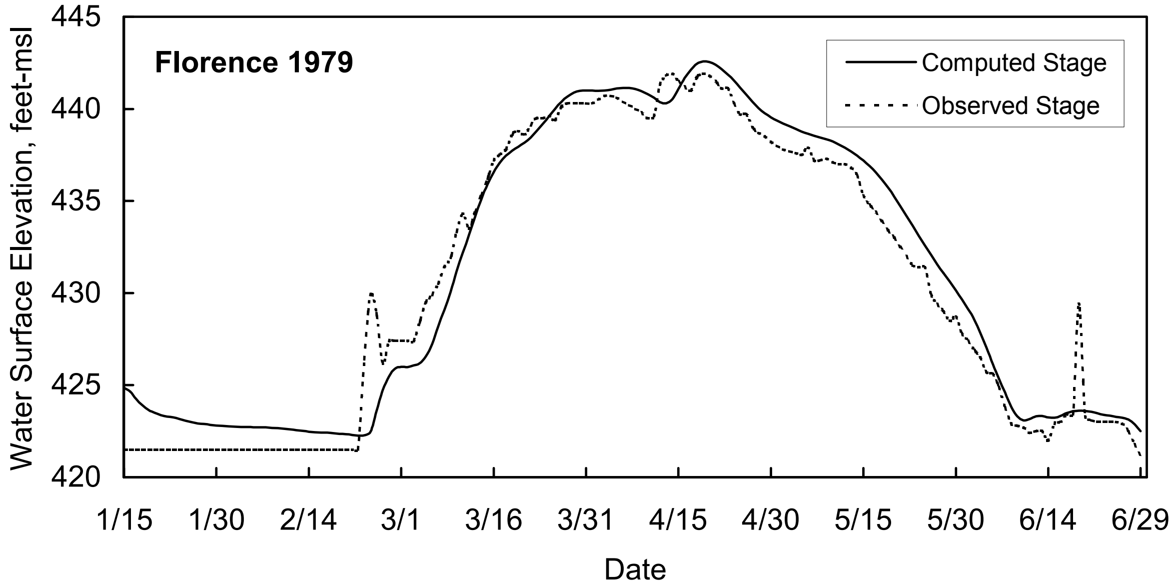

Figure 12. Computed and observed stage, Florence, 1979

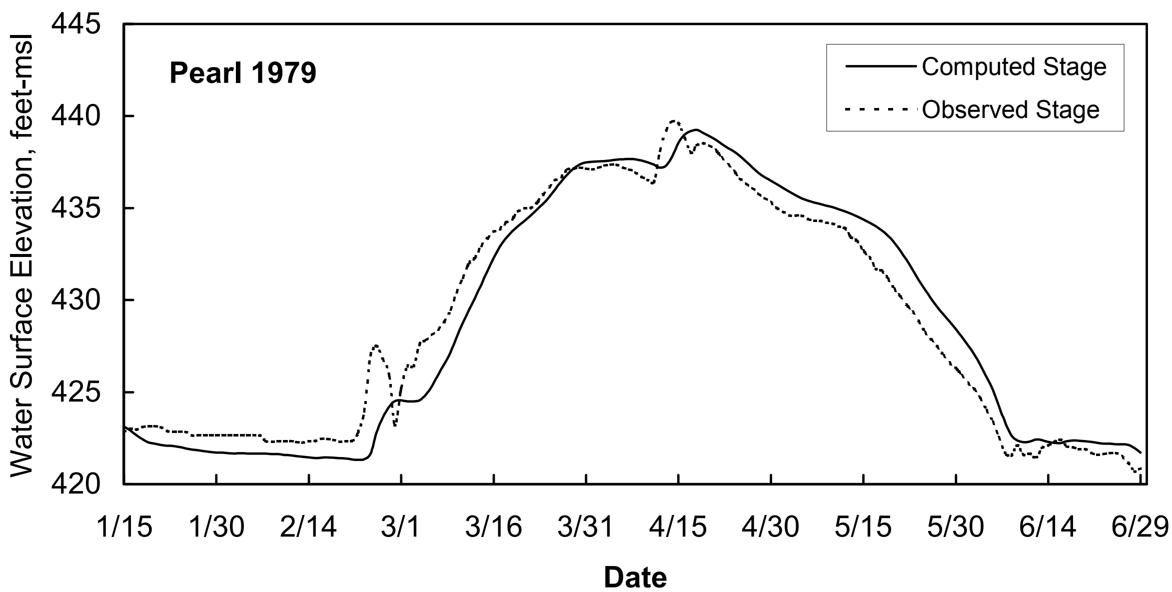

Figure 13. Computed and observed stage, Pearl, 1979

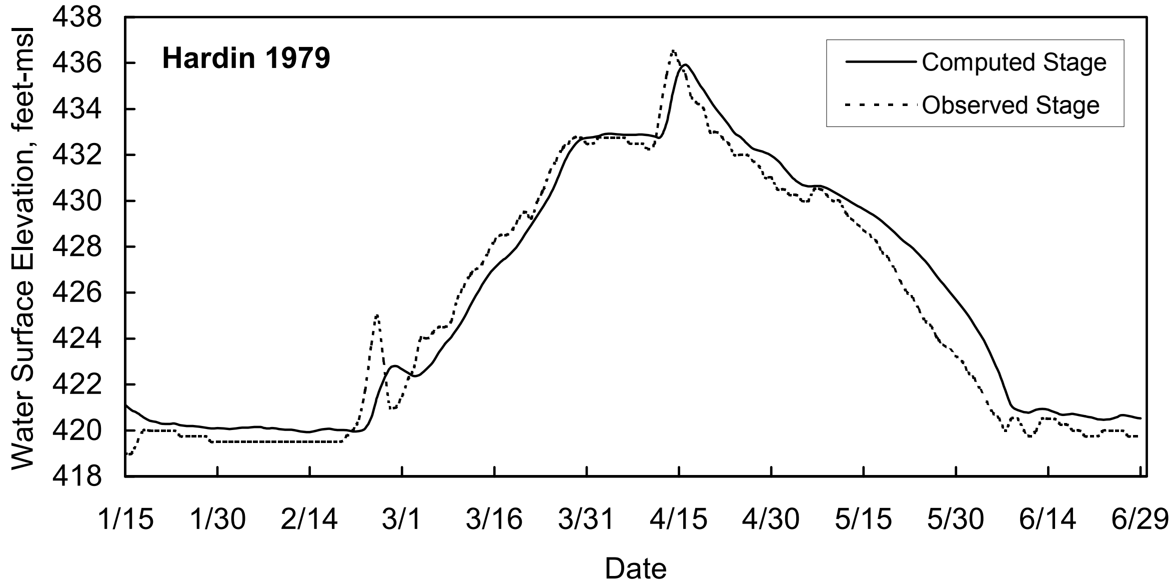

Figure 14. Computed and observed stage, Hardin, 1979

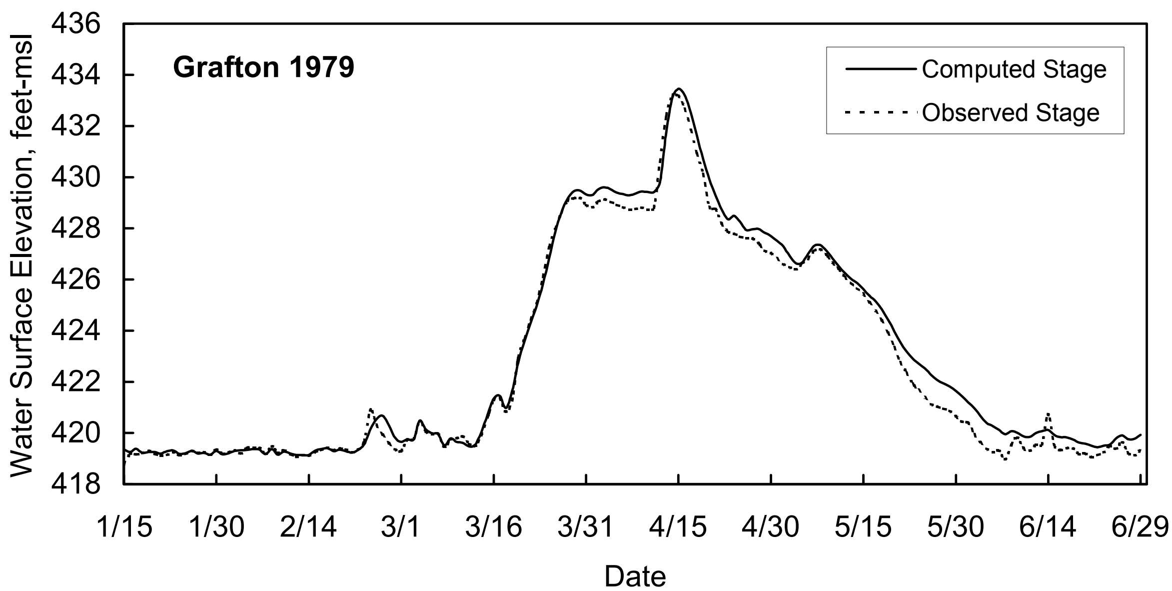

Figure 15. Computed and observed stage, Grafton, 1979

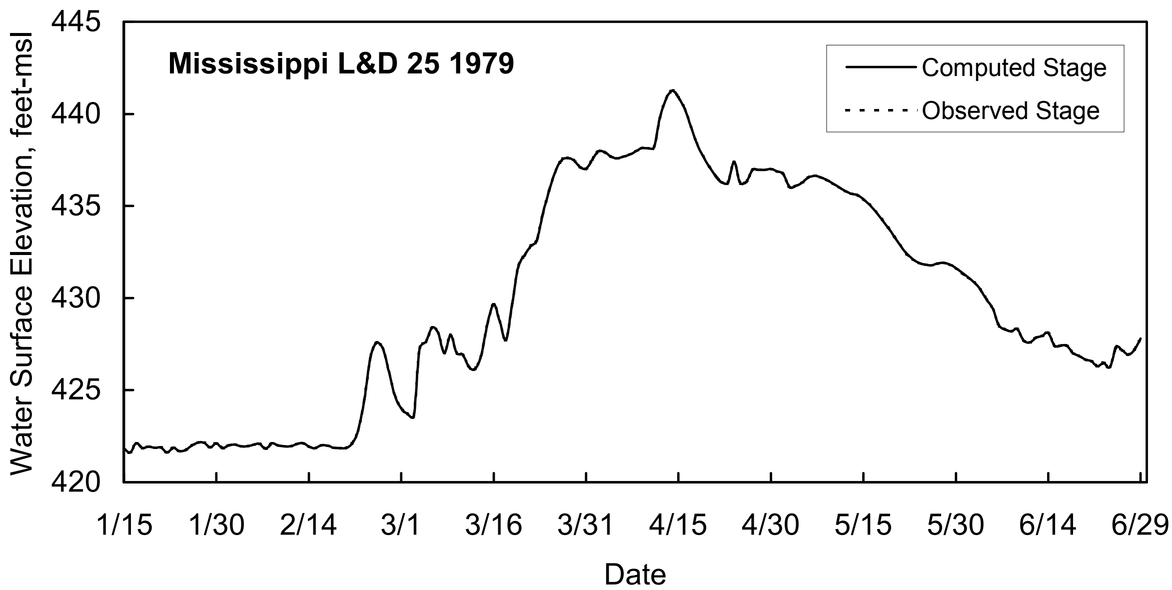

Figure 16. Computed and observed stage, Mississippi L&D 25, 1979

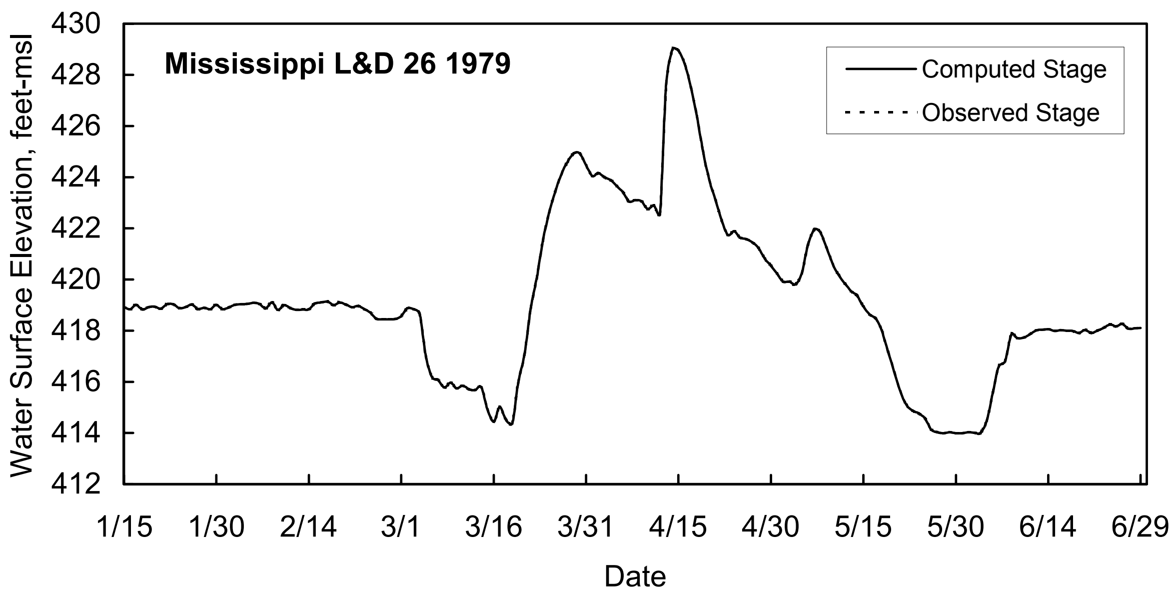

Figure 17. Computed and observed stage, Mississippi L&D 26, 1979

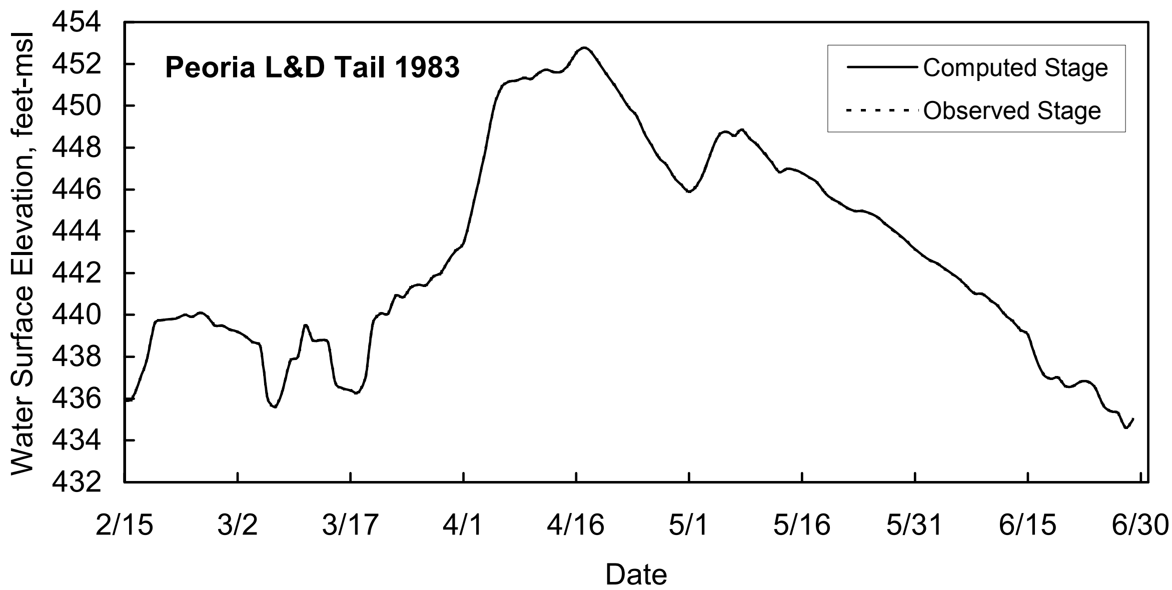

Figure 18. Computed and observed stage, Peoria Lock and Dam tail, 1983

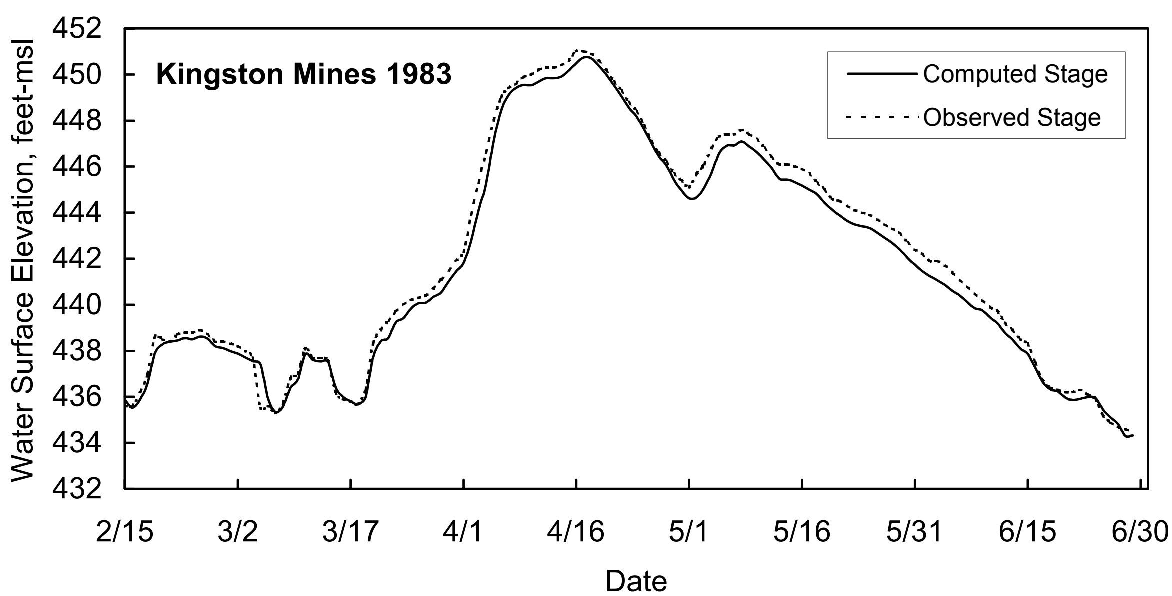

Figure 19. Computed and observed stage, Kingston Mines, 1983

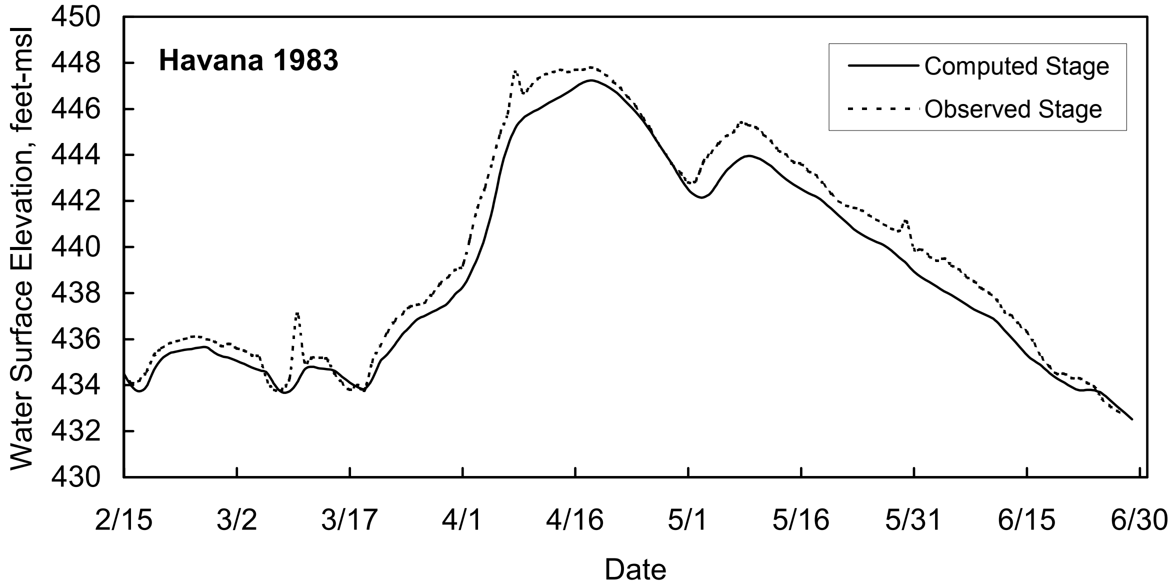

Figure 20. Computed and observed stage, Havana, 1983

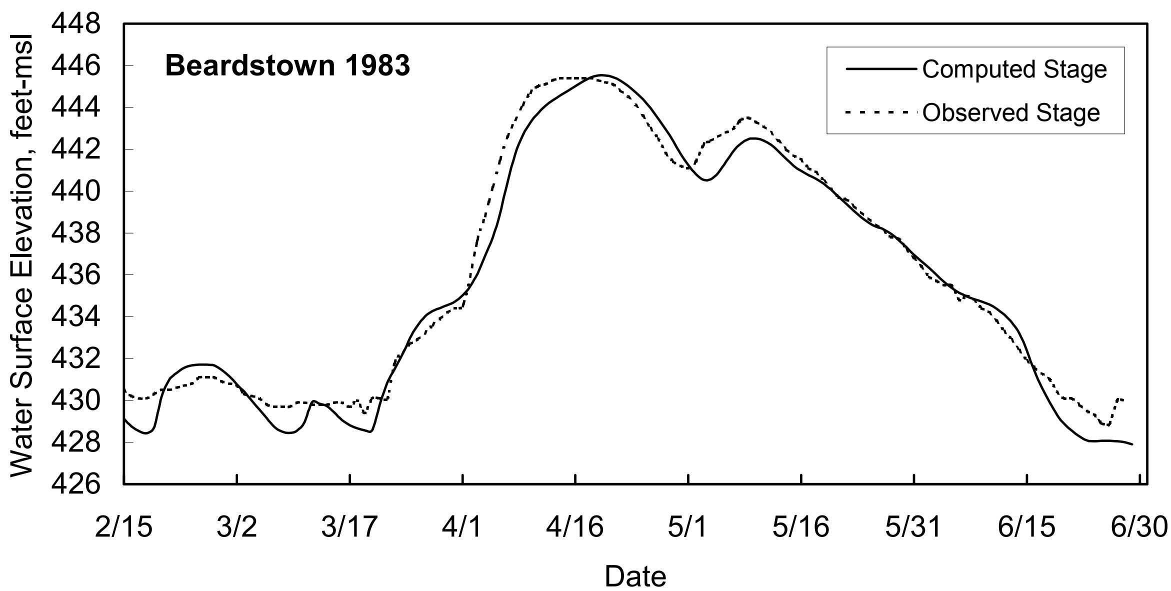

Figure 21. Computed and observed stage, Beardstown, 1983

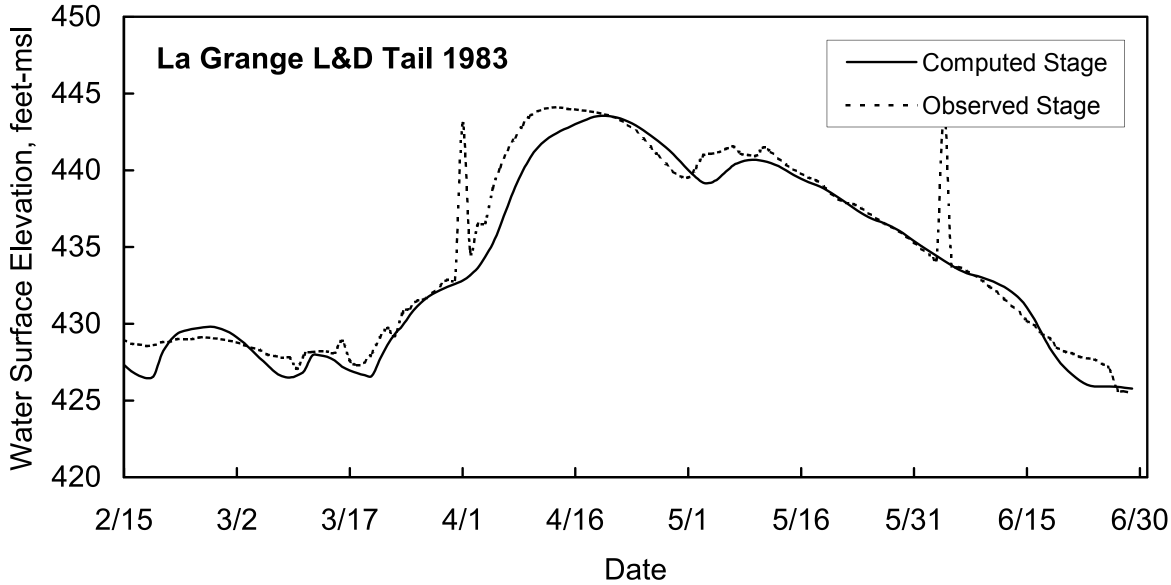

Figure 22. Computed and observed stage, La Grange, 1983

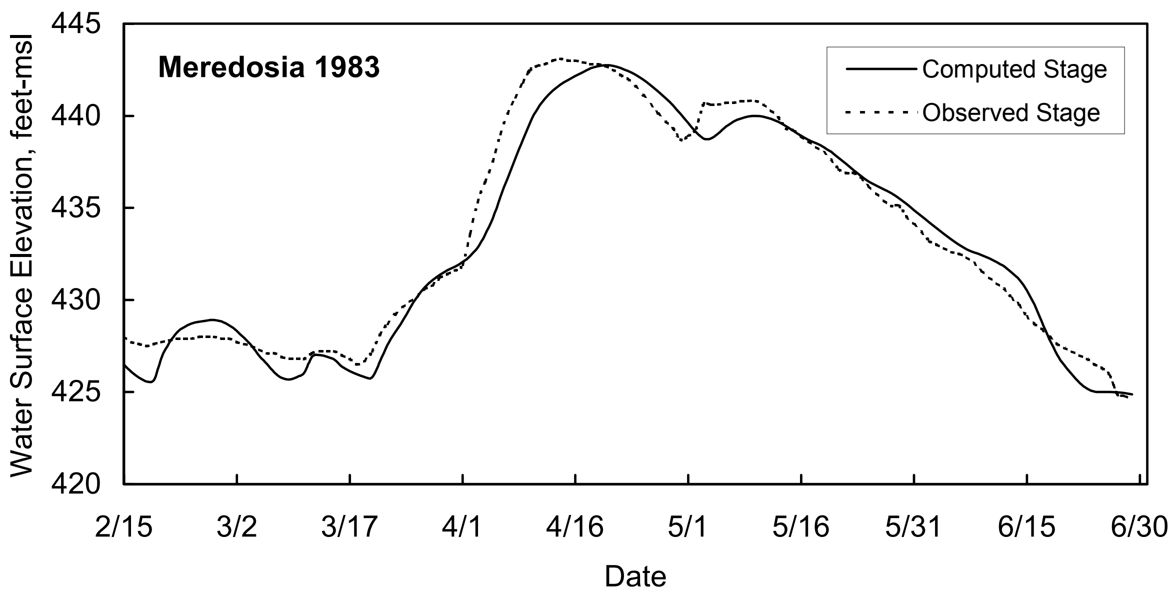

Figure 23. Computed and observed stage, Meredosia, 1983

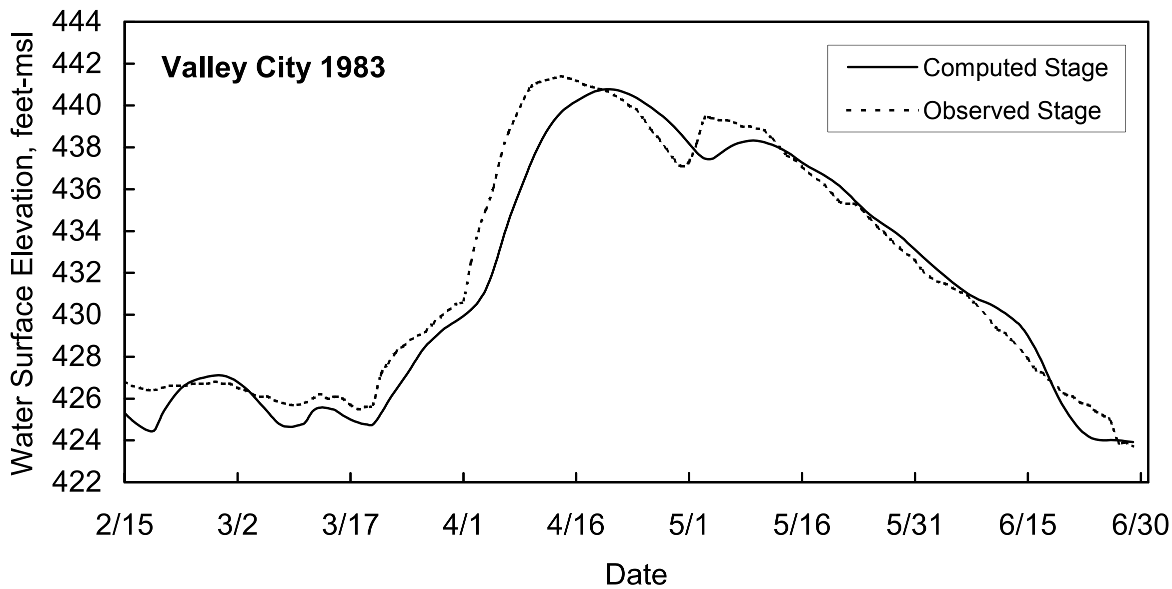

Figure 24. Computed and observed stage, Valley City, 1983

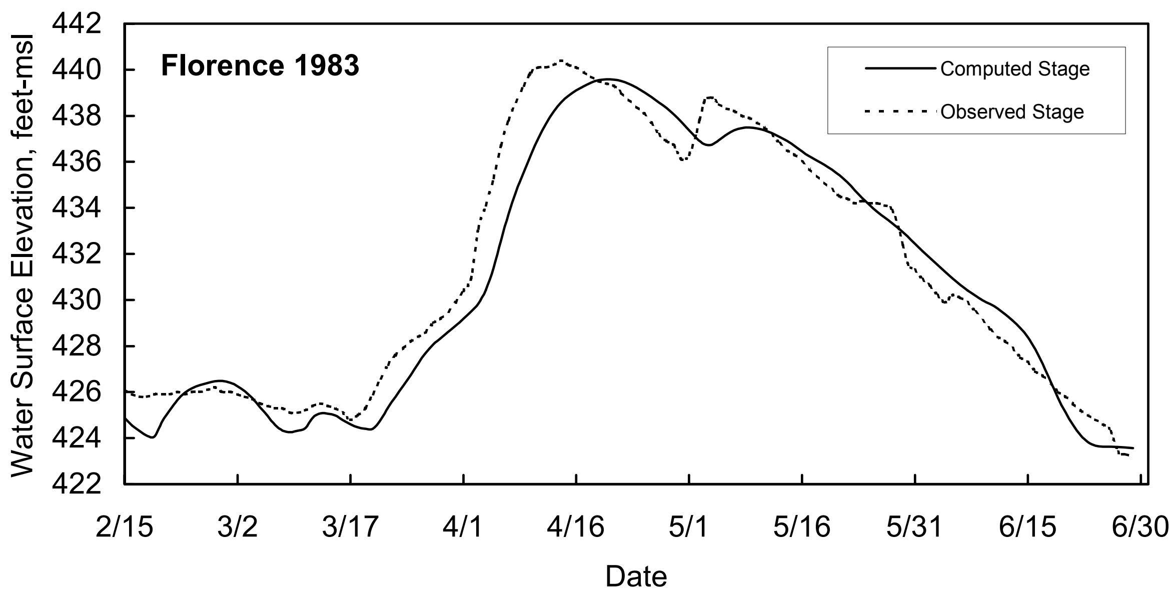

Figure 25. Computed and observed stage, Florence, 1983

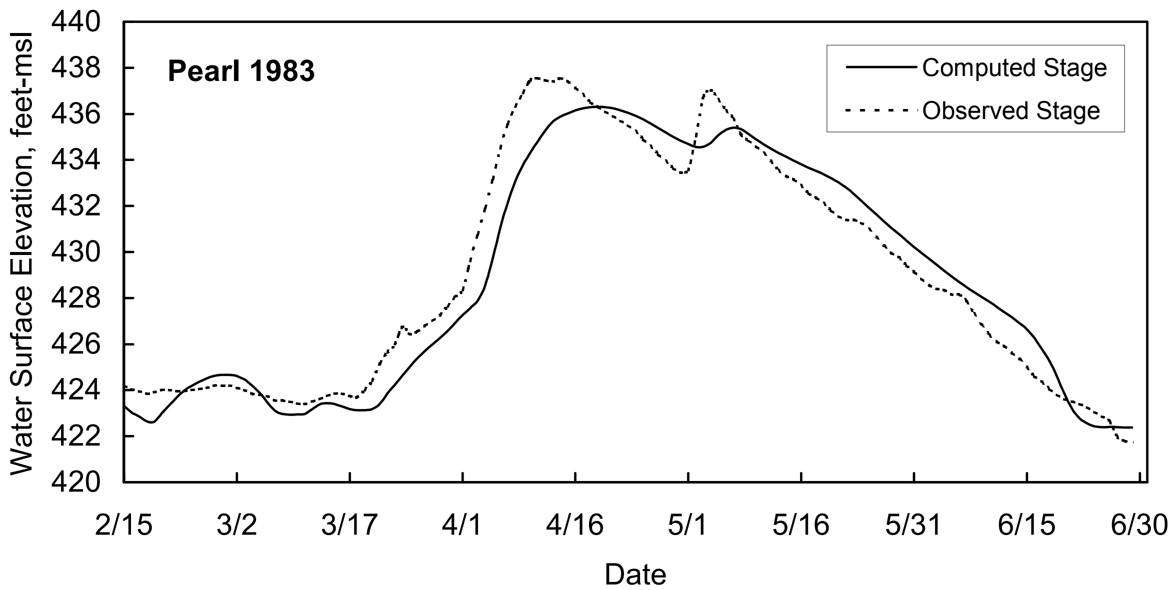

Figure 26. Computed and observed stage, Pearl, 1983

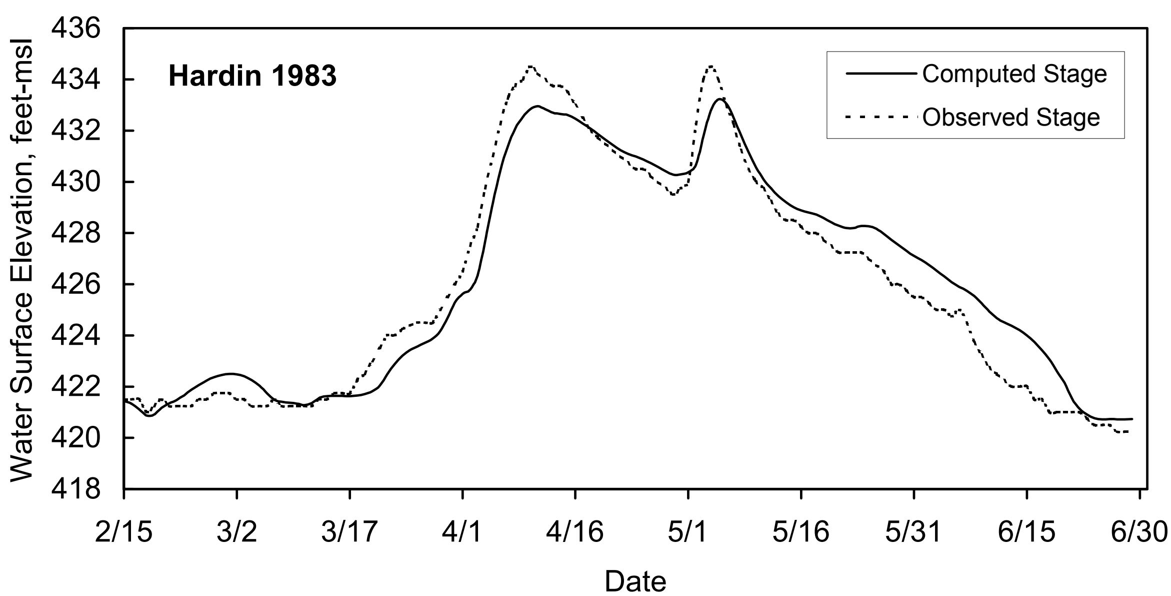

Figure 27. Computed and observed stage, Hardin, 1983

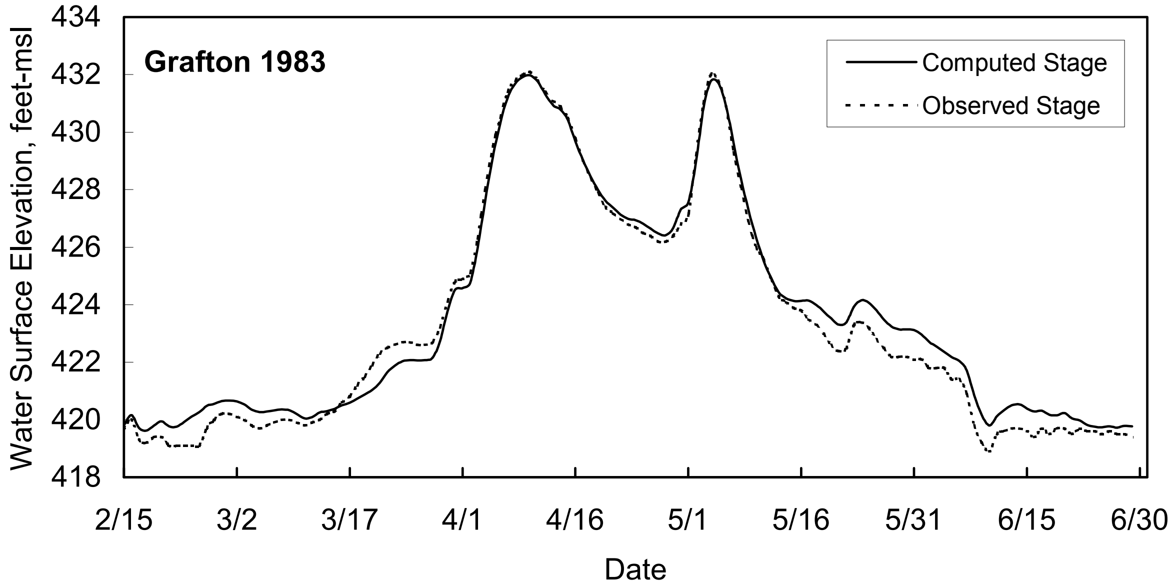

Figure 28. Computed and observed stage, Grafton, 1983

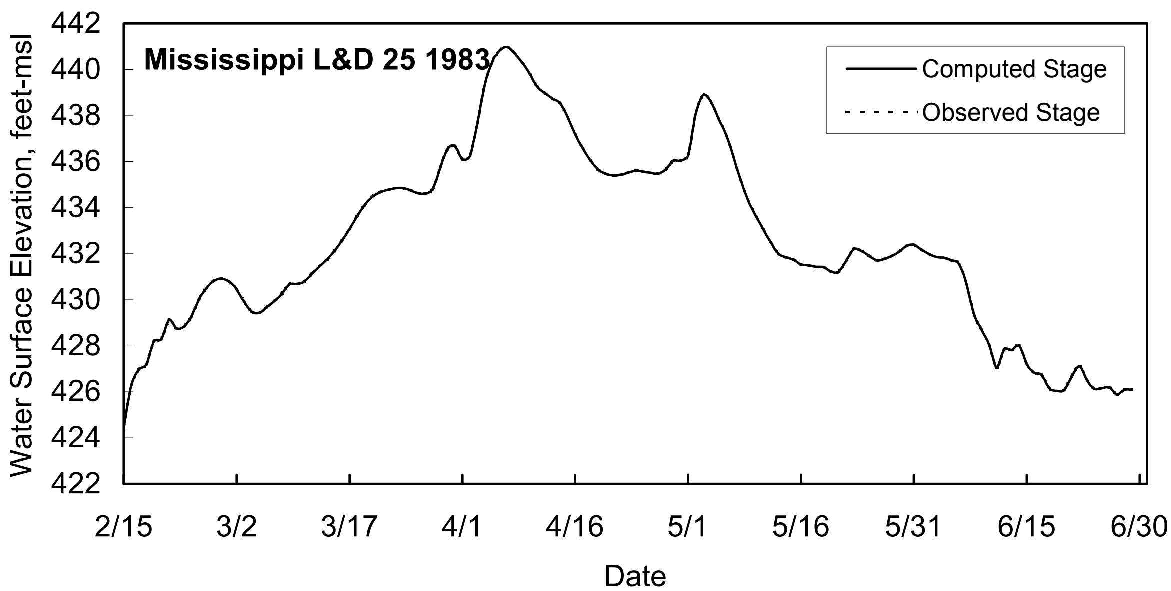

Figure 29. Computed and observed stage, Mississippi L&D 25, 1983

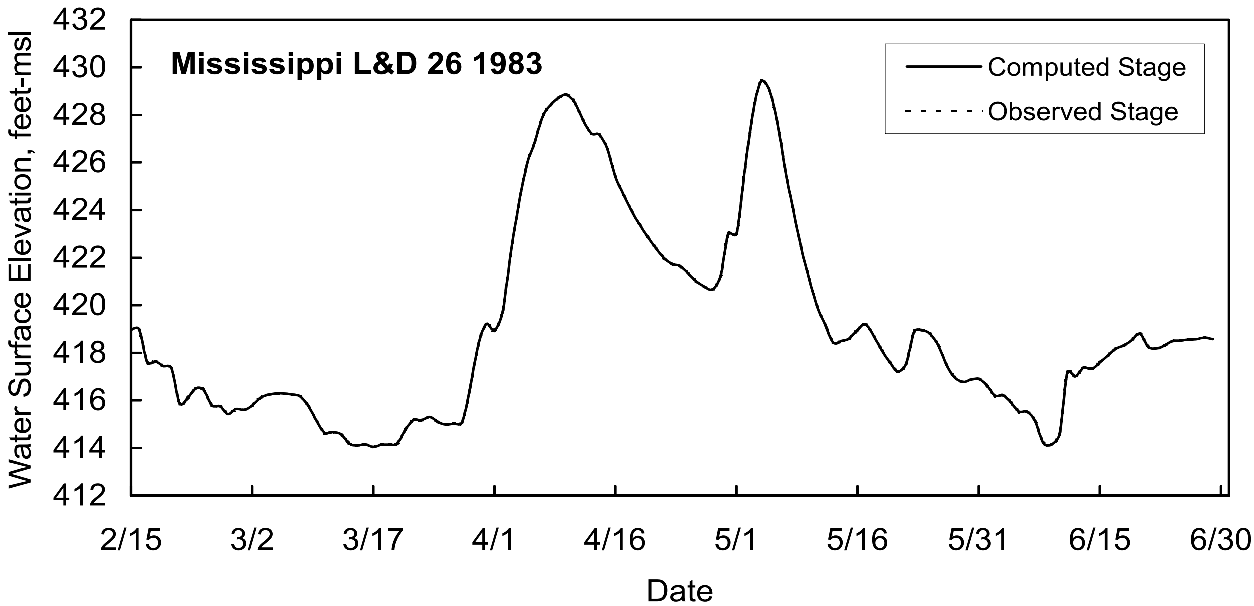

Figure 30. Computed and observed stage, Mississippi L&D 26, 1983

Frequency Analysis

Designating peak flood or stage at the boundary stations of the model for design purposes requires frequency analysis. The U.S. Water Resources Council (1973) recommended the Log-Pearson III (LP3) distribution for peak discharge analysis. Singh (1996) used a mixed mode distribution in addition to the LP3 to analyze flows and/or stages for gaging stations in the 1999 Lower Illinois River UNET model. Since the Lower Illinois River - Pool 26 UNET model is an extension of the former work, it is reasonable to use the same procedures to analyze the three new stations in the model: L&D 25 (tail water elevation), Cuivre River at Troy, and Mel Price L&D (pool elevation). Note that the Mel Price L&D was completed in 1988. Table 4 lists the available type of data and associated information for these stations.

The St. Louis District, Corps of Engineers provided historical data for stages at L&D 25 (tail), Mel Price L&D (pool), L&D 26 (pool), and discharge for the Cuivre River at Troy where annual peak values are obtained for each water year. Available data for Mel Price L&D contains only 11 years (including year 2000), barely enough for a frequency analysis. However, even though the distance between the stations for Mel Price L&D and L&D 26 is only 0.6 miles, we did not attempt to combine these two records for stage frequency analysis because of the differences in datum.

After examining various time period in searching for representative statistics in the frequency analysis, we selected the time periods of 1939-1995 for L&D 25 tail, 1980-1998 for Cuivre River at Troy, and 1980-95 for L&D 26 pool, and the whole period for Mel Price L&D pool. Results for these stations are shown in Tables 5-8, respectively.

Table 4. Available Stage or Flow Data for Stations in Pool 26 of the Mississippi River

Station	*Type of data*	*Period*	*Location*	*Datum, ft*
L&D 25 - tail	Stage	1939-current	RM 241.2	407.00
Cuivre River at Troy	Discharge	1923-1972 1978-1998	RM 39.9	
Mel Price - pool	Stage	1990-current	RM 201.1	395.48
L&D 26 - pool	Stage	1939-1990	RM 200.5	400.00

Results from L&D 25 tail are used as the example for illustrating the output. Table 5 is the output from a computer program developed in the previous study (Singh, 1996). The left-hand-side from the top shows the station number, name, drainage area, and the years used in the analysis (1939-1995). The level number corresponds to windows 0 to 6, where 0 represents no consideration of any outlier or inliers, and windows 1 - 6 represent a significance level of 0.01, 0.05, 0.10, 0.20, 0.30, and 0.40, respectively. As discussed in Singh (1996), the objective detection and modification of any outliers and inliers at various significance levels or windows is reflected in the change of values of high and low floods. A sub-table lists the estimated 100-year flood stages at various outliers/inliers modified according to three methods: power transform (PT), LP3, and mixed mode distributions (MD).

For the L&D 25 tail water, five detected significant outliers or inliers (if any) are given in the next sub-table. Subsequently modified values are given in the various windows.

Also given in the left-hand-side of Table 5 are the statistical parameters calculated for the three tested methods. For PT, skews approximate zero and kurtosis approximates 3, indicating the stage-frequency curve can be a sharper peaked distribution close to the normal distribution. Kite (1977) showed that all odd central moments are zero for a normal distribution. The fifth moment is generally positive, indicating that the transformed series is not symmetrical.

The 100-year peak stages sub-table discussed above are obtained for PT with kt = 3.0 (assumed PT series as a normal distribution) and with sample kt (allowing for correction but considering power-transformed series as a symmetric distribution), for LP3 with sample skew and weighted skew, assuming a regional skew of -0.4, and for MD. Although PT (sample kt) values are slightly lower than PT (kt = 3.0) values, the 100-year stage values generally stayed in a similar range with PT (kt = 3.0) and with PT (sample kt). The 100-year stages predicted with LP3, however, are somewhat higher but slightly lower with MD, as compared to PT results. Also the predicted values in each window do not change significantly. The sample skew for LP3 varies from -0.431 to -0.525 for windows 0-6, fairly close to the regional skew (- 0.4).

The right-hand-side of Table 5 gives the peak stages at 2-, 10-, 25-, 50-, 100-, 500-, and 1000-year return intervals predicted by the three distributions, and again at the given significance

Table 5. Stage Frequency Analysis, L&D 25 Tail Water Elevation

STATION NO. 2410 Mississippi River at L&D25 tail
DRAINAGE AREA 14200.0 Sq Mi Years of Record 57 (1939-1995)

LEVEL NO

METHOD	0	1	2	3	4	5	6
	100-Year Stage in feet						
Power Transform, PT							
With kt = 3.0	38.56	38.56	38.59	38.66	38.77	38.77	38.55
With sample kt	38.26	38.26	38.28	38.33	38.44	38.44	38.21
Log Transform							
LP3, Sample skew	38.82	38.82	38.86	38.92	39.01	38.97	38.68
LP3, Weighted skew	38.90	38.90	38.93	38.98	39.07	39.10	39.04
Mixed Distrib., MD	37.88	37.88	37.89	38.28	38.41	38.52	38.76

Type	Nank	Observed and Modified Stages in feet						
Low	1*	19.95	19.95	19.95	19.70	19.13	18.60	17.87
	2*	20.09	20.09	20.09	20.09	20.09	20.07	19.59
	3*	20.15	20.15	20.15	20.15	20.15	20.15	20.29
	4*	20.21	20.21	20.21	20.26	20.63	20.88	21.06
	5*	20.68	20.68	20.70	20.98	21.30	21.52	21.68
High	5*	33.77	33.77	33.77	33.79	34.12	34.34	34.51
	4*	33.98	33.98	34.00	34.27	34.62	34.87	35.06
	3*	34.25	34.25	34.55	34.84	35.24	35.51	35.73
	2*	36.93	36.93	36.93	36.93	36.93	36.93	36.93
	1*	39.57	39.57	39.57	39.57	39.57	39.40	38.70

Method	Statistics	Values of Statistics						
PT	mean	37.320	37.320	36.391	34.782	33.179	36.084	53.960
	std dev	6.831	6.831	6.618	6.251	5.903	6.613	11.145
	skew	-0.033	-0.033	-0.033	-0.033	-0.031	-0.032	-0.040
	kurtosis,kt	2.772	2.772	2.764	2.756	2.755	2.756	2.731
	5th moment	1.060	1.060	1.029	0.957	0.834	0.669	0.313
	lambda	1.125	1.125	1.115	1.097	1.078	1.111	1.267
LP3	mean	1.446	1.446	1.446	1.446	1.446	1.446	1.446
	std dev	0.072	0.072	0.072	0.072	0.072	0.072	0.073
	sample skew	-0.431	-0.431	-0.427	-0.423	-0.422	-0.445	-0.525
	kurtosis,kt	2.683	2.683	2.680	2.687	2.728	2.801	2.970
	5th moment	-2.042	-2.042	-2.025	-2.049	-2.207	-2.608	-3.695
MD	weight 'a'	0.389	0.389	0.389	0.213	0.209	0.209	0.208
	mu1	1.378	1.378	1.378	1.339	1.339	1.340	1.342
	mu2	1.488	1.488	1.488	1.474	1.474	1.474	1.473
	sigma1	0.055	0.055	0.055	0.034	0.035	0.040	0.044
	sigma2	0.042	0.042	0.042	0.049	0.049	0.050	0.052
	Test Stat	4.344	4.344	4.254	3.474	2.708	2.609	2.635

*High & low floods considered for outlier detection and modification

STATION NO. 2410 Mississippi River at L&D25 tail
DRAINAGE AREA 14200.0 Sq Mi Years of Record 57 (1939-1995)

VARIOUS RECURRENCE-INTERVAL FLOODS

Method	Level	Stages in feet for Recurrence Intervals (Years) 2	10	25	50	100	500	1000
PT, kt=3.0	0	28.30	34.00	36.05	37.37	38.56	40.93	41.84
PT, sample kt		28.30	34.06	35.99	37.20	38.26	40.36	41.14
LP3, sample skew		28.22	34.14	36.27	37.62	38.82	41.17	42.04
weighted skew		28.21	34.15	36.31	37.68	38.90	41.31	42.21
MD, mixed dist.		28.64	33.87	35.66	36.82	37.88	40.10	40.97
PT, kt=3.0	1							
PT, sample kt								
LP3, sample skew				SAME AS ABOVE				
weighted skew								
MD, mixed dist.								
PT, kt=3.0	2	28.31	34.02	36.08	37.41	38.59	40.98	41.90
PT, sample kt		28.31	34.08	36.01	37.22	38.28	40.38	41.17
LP3, sample skew		28.23	34.15	36.29	37.65	38.86	41.22	42.11
weighted skew		28.21	34.17	36.33	37.71	38.93	41.35	42.25
MD, mixed dist.		28.64	33.87	35.66	36.82	37.89	40.10	40.97
PT, kt=3.0	3	28.31	34.05	36.13	37.46	38.66	41.07	42.00
PT, sample kt		28.31	34.12	36.06	37.27	38.33	40.44	41.23
LP3, sample skew		28.23	34.18	36.34	37.70	38.92	41.30	42.20
weighted skew		28.22	34.20	36.37	37.75	38.98	41.41	42.32
MD, mixed dist.		28.68	33.87	35.80	37.09	38.28	40.79	41.79
PT, kt=3.0	4	28.33	34.11	36.21	37.56	38.77	41.21	42.15
PT, sample kt		28.33	34.18	36.14	37.36	38.44	40.57	41.37
LP3, sample skew		28.25	34.24	36.41	37.78	39.01	41.41	42.31
weighted skew		28.24	34.25	36.44	37.83	39.07	41.52	42.43
MD, mixed dist.		28.68	33.93	35.89	37.20	38.41	40.96	41.97
PT, kt=3.0	5	28.35	34.13	36.22	37.56	38.77	41.19	42.12
PT, sample kt		28.35	34.20	36.15	37.37	38.44	40.55	41.35
LP3, sample skew		28.28	34.26	36.41	37.77	38.97	41.33	42.20
weighted skew		28.26	34.28	36.47	37.86	39.10	41.53	42.44
MD, mixed dist.		28.66	33.97	35.96	37.29	38.52	41.11	42.15
PT, kt=3.0	6	28.39	34.09	36.11	37.40	38.55	40.85	41.73
PT, sample kt		28.39	34.16	36.04	37.19	38.21	40.19	40.92
LP3, sample skew		28.32	34.24	36.29	37.57	38.68	40.82	41.59
weighted skew		28.27	34.30	36.46	37.83	39.04	41.39	42.27
MD, mixed dist.		28.57	34.04	36.10	37.48	38.76	41.45	42.53

Table 6. Stage Frequency Analysis, L&D 26 Pool Water Elevation

1

STATION NO. 2011 Mississippi River at L&D26 - Pool
DRAINAGE AREA 171500.0 Sq Mi Years of Record 19 (1970-1988)

Level No	0	1	2	3	4	5	6
METHOD				100-Year Stage in feet			
Power Transform, PT							
With kt = 3.0	58.23	58.23	57.86	59.20	52.93	50.40	50.15
With sample kt	40.02	40.02	40.03	40.68	40.67	41.02	43.09
Log Transform							
LP3, Sample skew	37.93	37.93	37.93	37.92	37.90	37.88	38.92
LP3, Weighted skew	32.44	32.44	32.44	32.47	32.51	32.55	32.96
Mixed Distrib., MD	36.32	36.32	36.51	37.37	37.83	36.27	37.73

Type	Rank	Observed and Modified Stages in feet						
Low	1*	18.99	18.99	18.96	18.75	18.45	18.20	17.93
	2	19.16						
	3	19.21						
	4	19.27						
	5	19.70						
High	5	26.07						
	4	29.00						
	3	29.40						
	2	30.30						
	1*	33.13	33.13	33.12	33.13	33.12	33.12	34.53

METHOD	STATISTICS	Values of Statistics						
PT	mean	0.372	0.372	0.373	0.375	0.400	0.420	0.441
	std dev	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	skew	0.218	0.218	0.216	0.184	0.173	0.149	0.146
	kurtosis,kt	2.155	2.155	2.159	2.178	2.247	2.319	2.465
	5th moment	1.241	1.241	1.232	1.042	0.911	0.681	0.621
	lambda	-2.688	-2.688	-2.678	-2.669	-2.499	-2.377	-2.266
LP3	mean	1.362	1.362	1.362	1.362	1.361	1.361	1.362
	std dev	0.076	0.076	0.076	0.076	0.077	0.077	0.080
	sample skew	0.747	0.747	0.745	0.733	0.711	0.689	0.760
	kurtosis,kt	2.820	2.820	2.818	2.814	2.808	2.807	3.107
	5th moment	4.864	4.864	4.849	4.770	4.620	4.472	5.619
MD	weight 'a'	0.315	0.315	0.315	0.315	0.315	0.521	0.547
	mu1	1.369	1.369	1.374	1.392	1.404	1.412	1.405
	mu2	1.383	1.383	1.379	1.360	1.348	1.305	1.309
	sigma1	0.097	0.097	0.098	0.096	0.093	0.071	0.082
	sigma2	0.064	0.064	0.065	0.064	0.061	0.031	0.032
	Test Stat	0.017	0.017	0.015	0.008	0.004	1.794	1.726

*High & low floods considered for outlier detection and modification

1

STATION NO. 2011 Mississippi River at L&D26 - Pool
DRAINAGE AREA 171500.0 Sq Mi Years of Record 19 (1970-1988)

Various Recurrence-Interval Flood Stages

METHOD	Level	Stage in feet for Recurrence Intervals (Years) 2	10	25	50	100	500	1000
PT, kt=3.0	0	22.23	28.97	34.61	41.77	58.23	0.00	0.00
PT, sample kt		22.23	29.48	33.29	36.43	40.02	52.51	62.64
LP3, sample skew		22.52	29.07	32.54	35.20	37.93	44.65	47.73
weighted skew		23.34	28.48	30.29	31.43	32.44	34.38	35.09
MD, mixed dist.		23.99	29.76	32.38	34.33	36.32	41.14	43.28
PT, kt=3.0	1							
PT, sample kt								
LP3, sample skew					SAME AS ABOVE			
weighted skew								
MD, mixed dist.								
PT, kt=3.0	2	22.23	28.97	34.60	41.71	57.86	0.00	0.00
PT, sample kt		22.23	29.47	33.29	36.43	40.03	52.57	62.79
LP3, sample skew		22.52	29.07	32.54	35.20	37.93	44.64	47.72
weighted skew		23.34	28.48	30.29	31.44	32.44	34.38	35.09
MD, mixed dist.		23.86	29.68	32.39	34.43	36.51	41.55	43.76
PT, kt=3.0	3	22.21	29.04	34.77	42.08	59.20	0.00	0.00
PT, sample kt		22.21	29.53	33.48	36.80	40.68	55.21	68.76
LP3, sample skew		22.51	29.07	32.54	35.20	37.92	44.61	47.68
weighted skew		23.33	28.49	30.31	31.46	32.47	34.42	35.14
MD, mixed dist.		23.30	29.33	32.47	34.91	37.37	42.92	45.23
PT, kt=3.0	4	22.22	29.05	34.49	40.88	52.93	0.00	0.00
PT, sample kt		22.22	29.48	33.45	36.78	40.67	54.88	67.20
LP3, sample skew		22.50	29.08	32.55	35.19	37.90	44.53	47.57
weighted skew		23.31	28.50	30.34	31.49	32.51	34.48	35.21
MD, mixed dist.		22.92	29.17	32.66	35.29	37.83	43.36	45.63
PT, kt=3.0	5	22.23	29.09	34.37	40.29	50.40	0.00	0.00
PT, sample kt		22.23	29.48	33.51	36.95	41.02	56.26	69.78
LP3, sample skew		22.50	29.10	32.55	35.19	37.88	44.45	47.46
weighted skew		23.29	28.51	30.36	31.53	32.55	34.54	35.28
MD, mixed dist.		21.89	29.80	32.64	34.53	36.27	39.98	41.58
PT, kt=3.0	6	22.25	29.29	34.64	40.51	50.15	0.00	0.00
PT, sample kt		22.25	29.59	34.03	38.03	43.09	66.29	100.07
LP3, sample skew		22.47	29.39	33.10	35.96	38.92	46.24	49.63
weighted skew		23.34	28.76	30.68	31.90	32.96	35.03	35.79
MD, mixed dist.		21.94	30.14	33.44	35.66	37.73	42.21	44.16

Table 7. Discharge Frequency Analysis, Cuivre River at Troy

1

STATION NO. 5514500 Cuivre River at Troy
DRAINAGE AREA 903.0 Sq Mi Years of Record 19 (1980-1998)

LEVEL NO.	0	1	2	3	4	5	6
METHOD				100-Year Flood in cfs			
Power Transform, PT							
With kt = 3.0	133817.	133817.	133817.	133817.	136184.	177196.	251623.
With sample kt	135324.	135324.	135324.	135324.	138216.	168877.	217518.
Log Transform							
LP3, Sample skew	134233.	134233.	134233.	134233.	136408.	157954.	182897.
LP3, Weighted skew	107003.	107003.	107003.	107003.	107811.	104972.	107342.
Mixed Distrib., MD	134791.	134791.	134791.	134791.	135153.	134235.	138859.

Type	Rank				Observed and Modified Floods in cfs			
Low	1*	4920.	4920.	4920.	4920.	4920.	6664.	7889.
	2	11300.						
	3	12800.						
	4	13500.						
	5	13900.						
High	5	50300.						
	4	59600.						
	3	72000.						
	2	72200.						
	1*	74100.	74100.	74100.	74100.	76833.	90105.	110960.

METHOD	STATISTICS				Values of Statistics			
PT	mean	10.678	10.678	10.678	10.678	10.303	4.288	2.697
	std dev	0.828	0.828	0.828	0.828	0.775	0.092	0.018
	skew	0.000	0.000	0.000	0.000	0.001	0.042	0.092
	kurtosis,kt	3.057	3.057	3.057	3.057	3.074	2.848	2.701
	5th moment	-1.933	-1.933	-1.933	-1.933	-1.931	-1.050	-0.214
	lambda	0.011	0.011	0.011	0.011	0.004	-0.203	-0.361
LP3	mean	4.384	4.384	4.384	4.384	4.384	4.395	4.404
	std dev	0.322	0.322	0.322	0.322	0.323	0.315	0.318
	sample skew	-0.019	-0.019	-0.019	-0.019	-0.006	0.308	0.524
	kurtosis,kt	3.083	3.083	3.083	3.083	3.084	2.695	2.792
	5th moment	-2.116	-2.116	-2.116	-2.116	-1.998	1.135	3.242
MD	weight 'a'	1.000	1.000	1.000	1.000	1.000	1.000	1.000
	mu1	4.384	4.384	4.384	4.384	4.385	4.395	4.404
	mu2	2.480	2.480	2.480	2.480	2.480	2.480	2.480
	sigma1	0.321	0.321	0.321	0.321	0.321	0.315	0.318
	sigma2	0.005	0.005	0.005	0.005	0.005	0.005	0.005
	Test Stat	0.000	0.000	0.000	0.000	0.002	0.012	0.019

*High & low floods considered for outlier detection and modification

1

STATION NO. 5514500 Cuivre River at Troy
DRAINAGE AREA 903.0 Sq Mi Years of Record 19 (1980-1998)

Various Recurrence-Interval Floods

METHOD	Level	2	10	25	50	100	500	1000
		Flood in cfs for Recurrence Intervals (Years)						
PT, kt=3.0	0	24252.	62387.	87954.	109726.	133817.	199724.	232849.
PT, sample kt		24252.	62228.	88155.	110469.	135324.	204308.	239333.
LP3, sample skew		24239.	62423.	88088.	109979.	134233.	200729.	234214.
weighted skew		25500.	60024.	78815.	92935.	107003.	139454.	153311.
MD, mixed dist.		24201.	62315.	88088.	110228.	134791.	202557.	236945.
PT, kt=3.0	1							
PT, sample kt								
LP3, sample skew					SAME AS ABOVE			
weighted skew								
MD, mixed dist.								
PT, kt=3.0	2							
PT, sample kt								
LP3, sample skew					SAME AS ABOVE			
weighted skew								
MD, mixed dist.								
PT, kt=3.0	3							
PT, sample kt								
LP3, sample skew					SAME AS ABOVE			
weighted skew								
MD, mixed dist.								
PT, kt=3.0	4	24254.	62839.	88956.	111324.	136184.	204697.	239366.
PT, sample kt		24254.	62622.	89217.	112307.	138216.	210930.	248229.
LP3, sample skew		24247.	62856.	89027.	111458.	136408.	205249.	240119.
weighted skew		25557.	60359.	79331.	93594.	107811.	140616.	154629.
MD, mixed dist.		24247.	62477.	88356.	110551.	135153.	203103.	237576.
PT, kt=3.0	5	23618.	64683.	99158.	133461.	177196.	332414.	433242.
PT, sample kt		23618.	65213.	98299.	129931.	168877.	298805.	378063.
LP3, sample skew		23919.	64226.	95180.	123858.	157954.	263211.	322410.
weighted skew		26231.	60283.	78355.	91757.	104972.	134974.	147595.
MD, mixed dist.		24840.	62907.	88440.	110152.	134235.	200423.	233756.
PT, kt=3.0	6	23204.	66955.	112412.	166851.	251623.	743233.	*******
PT, sample kt		23204.	68225.	109723.	154607.	217518.	497150.	732125.
LP3, sample skew		23762.	66730.	102961.	138510.	182897.	331832.	422201.
weighted skew		26825.	61827.	80298.	93940.	107342.	137582.	150223.
MD, mixed dist.		25306.	64678.	91160.	113743.	138859.	207961.	242850.

Table 8. Stage Frequency Analysis, Mel Price L&D Pool

1

STATION NO. 2010 Mississippi River at Mel Price - Pool
DRAINAGE AREA 171500.0 Sq Mi Years of Record 11 (1990-2000)

LEVEL NO.	0	1	2	3	4	5	6
METHOD	100-Year Flood Stage in feet						
Power Transform, PT							
With kt = 3.0	61.47	61.47	61.47	61.47	62.71	58.85	50.98
With sample kt	63.12	63.12	63.12	63.12	64.34	60.85	51.85
Log Transform							
LP3, Sample skew	50.24	50.24	50.24	50.24	50.23	50.18	47.03
LP3, Weighted skew	40.91	40.91	40.91	40.91	40.95	41.09	40.44
Mixed Distrib., MD	47.47	47.47	47.47	47.47	49.70	50.06	46.99

Type	Rank	Observed and Modified Stages in feet						
Low	1*	23.80	23.80	23.80	23.80	23.66	23.23	22.78
	2	23.80						
	3	23.86						
	4	27.27						
	5	27.44						
High	5	28.71						
	4	30.35						
	3	33.09						
	2	35.65						
	1*	43.02	43.02	43.02	43.02	43.02	43.01	40.73

Method	Statistics	Values of Statistics						
PT	mean	0.459	0.459	0.459	0.459	0.454	0.505	0.660
	std dev	0.000	0.000	0.000	0.000	0.000	0.000	0.001
	skew	0.132	0.132	0.132	0.132	0.109	0.115	0.074
	kurtosis,kt	3.109	3.109	3.109	3.109	3.098	3.170	3.153
	5th moment	1.949	1.949	1.949	1.949	1.751	1.801	1.146
	lambda	-2.178	-2.178	-2.178	-2.178	-2.199	-1.979	-1.506
LP3	mean	1.463	1.463	1.463	1.463	1.463	1.463	1.460
	std dev	0.080	0.080	0.080	0.080	0.081	0.081	0.078
	sample skew	0.903	0.903	0.903	0.903	0.891	0.846	0.574
	kurtosis,kt	4.367	4.367	4.367	4.367	4.349	4.294	3.672
	5th moment	10.625	10.625	10.625	10.625	10.480	9.974	6.097
MD	weight 'a'	0.403	0.403	0.403	0.403	0.472	0.573	0.404
	mu1	1.383	1.383	1.383	1.383	1.425	1.428	1.425
	mu2	1.518	1.518	1.518	1.518	1.498	1.509	1.483
	sigma1	0.025	0.025	0.025	0.025	0.030	0.044	0.037
	sigma2	0.075	0.075	0.075	0.075	0.096	0.096	0.089
	Test Stat	0.001	0.001	0.001	0.001	1.909	1.523	1.405

*High & low floods considered for outlier detection and modification

1

STATION NO. 2010 Mississippi River at Mel Price - Pool
DRAINAGE AREA 171500.0 Sq Mi Years of Record 11 (1990-2000)

VARIOUS RECURRENCE-INTERVAL FLOOD Stages

Method	Rank	Stage in feet for Recurrence Intervals (Years) 2	10	25	50	100	500	1000
PT, kt=3.0	0	28.20	37.10	43.69	50.68	61.47	0.00	0.00
PT, sample kt		28.20	37.03	43.80	51.19	63.12	0.00	0.00
LP3, sample skew		28.28	37.24	42.23	46.15	50.24	60.60	65.49
weighted skew		29.65	36.25	38.44	39.78	40.91	43.03	43.77
MD, mixed dist.		27.87	38.86	42.60	45.12	47.47	52.54	54.64
PT, kt=3.0	1							
PT, sample kt								
LP3, sample skew		SAME AS ABOVE						
weighted skew								
MD, mixed dist.								
PT, kt=3.0	2							
PT, sample kt								
LP3, sample skew		SAME AS ABOVE						
weighted skew								
MD, mixed dist.								
PT, kt=3.0	3							
PT, sample kt								
LP3, sample skew		SAME AS ABOVE						
weighted skew								
MD, mixed dist.								
PT, kt=3.0	4	28.17	37.16	43.90	51.18	62.71	0.00	0.00
PT, sample kt		28.17	37.09	44.00	51.66	64.34	0.00	0.00
LP3, sample skew		28.28	37.25	42.23	46.15	50.23	60.55	65.41
weighted skew		29.63	36.26	38.46	39.81	40.95	43.08	43.83
MD, mixed dist.		27.78	38.19	43.15	46.52	49.70	56.76	59.73
		28.57	37.19	41.53	44.75	47.95	55.33	58.50
PT, kt=3.0	5	28.18	37.20	43.54	49.91	58.85	126.97	0.00
PT, sample kt		28.18	37.08	43.69	50.59	60.85	199.88	0.00
LP3, sample skew		28.26	37.28	42.25	46.14	50.18	60.34	65.11
weighted skew		29.58	36.29	38.54	39.92	41.09	43.27	44.04
MD, mixed dist.		28.04	37.90	43.19	46.74	50.06	57.28	60.26
PT, kt=3.0	6	28.21	36.62	41.65	45.97	50.98	67.23	77.89
PT, sample kt		28.21	36.54	41.75	46.35	51.85	70.92	84.65
LP3, sample skew		28.33	36.54	40.71	43.86	47.03	54.66	58.10
weighted skew		29.31	35.74	37.93	39.28	40.44	42.64	43.42
MD, mixed dist.		28.03	37.02	41.31	44.23	46.99	52.98	55.45

levels. Based on his experience, Singh (1996) indicated that an acceptable level could be taken as 0.10 or Level 3. The same methodologies have been used for stage frequency and for flood frequency analysis.

Comparison with MBM Results

The water surface profile derived from MBM studies provided an opportunity to examine the results of stage frequency analysis. Note that the MBM was conducted with fixed bed and steady state conditions; therefore the return period of resulting water surface profiles can correspond to the return periods specified to inflows and downstream stages. However, Foster (1979) noted that tributaries were adjusted to simulate input from smaller tributaries and verified with the stage-discharge relationship at Wickliffe, Kentucky, the downstream control station. When making such a comparison, keep in mind the uncertainties involved in the analysis, that the frequency analysis derived its values from a best-fit curve among scattered data points, and that there is significant approximation involved in building physical models. Nonetheless, it is informative to compare the stage information derived from experiments and from frequency analysis for stations within the study reaches. Thirty-five MBM tests were completed. Among them, tests 2-5 and 17 are useful for stations in Pool 26, and tests 25-28 and 32 are useful for

Table 9. Comparison Stage Results from MBM and Frequency Analysis

		Stage above msl, feet				
Location	*Analysis*	*10*	*25*	*50*	*100*	*500*
L&D 26	MBM	427.40	430.60	433.70	436.70	442.20
	FQ	429.20	432.50	435.05	437.20	443.76
Meredosia	MBM	441.60	442.20	446.60	448.60	451.80
	FQ	442.92	445.38	447.08	448.59	451.89
Valley City	MBM	440.20	443.00	445.10	447.20	449.90
	FQ	441.22	443.84	445.61	447.29	450.88
Florence	MBM	439.20	442.20	444.30	446.40	449.20
	FQ	440.19	442.85	444.66	446.35	449.97
Pearl	MBM	436.10	439.80	442.10	444.40	447.50
	FQ	437.44	440.15	442.07	443.91	448.01
Hardin	MBM	432.90	436.10	438.80	441.30	445.50
	FQ	434.55	437.49	439.53	441.47	445.71

Note:

The MBM test results used to obtain stages at L&D 26 were as follows: test 2 (10-year), test 3 (25-year), test 4 (50-year), test 5 (100-year), and test 17 (50-year). The MBM test results used to obtain stages on the Lower Illinois River were as follows: test 25 (10-year), test 26 (25-year), test 27 (50-year), test 28 (100-year), and test 32 (500-year).

Tests 1-16 (35 tests total) were conducted with the downstream control at Wickliffe; tests 17, 24-30, and 32 were conducted with the downstream control at Chain of Rocks with elevations recorded for tests 1-17.

Tests 1-8 were conducted with the existing levees on the Mississippi and Missouri Rivers and authorized levees on the Illinois River. Tests 24-35 were conducted with the authorized levees installed except for the combined Kaskaskia Island-Ste. Genevieve area levee.

stations in the Lower Illinois River. Comparisons of stage results from MBM and frequency analysis are shown (Table 9). One finds that frequency analysis generally produces higher stages than experimental results for floods of low return periods (less than 50 years) but generally lower stages at high return periods.

Geometric Data

During the development of the Lower Illinois River - Pool 26 UNET model, cross-sectional geometry for the current model (measured around 1978) was compared with a newer set (measured in 1995). Some discrepancies in terms of levee height and cross-sectional width were noted. The differences in width were verified through comparison with the quadrangle topographic map (USGS) and by making a judgment. However, the differences in levee elevation can be measuring error of the instrument used or due to cross sections not lining up correctly.

Table 10. Differences in Levee Height (feet) Measured

from 1995 and 1978 Survey

River Mile	*1978*	*1995*	*1995-1978*	*River Mile*	*1978*	*1995*	*1995-1978*
4.80	426	427.57	1.57		*Eldred & Spankey LDD*		
5.10	426	428.79	2.79	24.26	438.5	440.26	1.76
5.80	426.2	427.25	1.05	24.70	438.5	441.30	2.80
6.80	no levee	424.97		25.10	438.5	439.91	1.41
8.70	no levee	430.19		25.51	438.5	440.02	1.52
11.80	425	425.50	0.50	25.88	441.6	440.20	-1.40
	Nutwood LDD			26.20	438.5	440.00	1.50
15.54	437	439.46	2.46	26.70	438.5	440.17	1.67
15.90	437	439.63	2.63	27.20	438.5	440.44	1.94
16.31	437	439.23	2.23	27.70	438.5	440.46	1.96
16.70	437	439.42	2.42	28.10	438.5	441.58	3.08
17.30	437	438.68	1.68	28.50	438.5	441.57	3.07
17.70	437	438.68	1.68	29.00	438.5	439.70	1.20
17.96	437	437.20	0.20	29.45	438.5	438.40	-0.10
18.25	437	437.34	0.34	29.83	438.5	438.16	-0.34
18.64	437	439.23	2.23	30.20	438.5	436.90	-1.60
19.10	437	438.06	1.06	30.50	438.5	438.97	0.47
19.40	437	437.89	0.89	30.80	438.5	438.86	0.36
19.58	437	438.35	1.35	31.30	438.5	440.07	1.57
20.20	437	436.47	-0.53	31.70	438.5	439.73	1.23
20.50	437	437.44	0.44	31.90	438.5	440.00	1.50
21.00	437	437.88	0.88	32.30	438.5	no levee	
21.67	437	437.33	0.33				
22.05	437	437.9	0.9				
22.60	437	437.67	0.67				
23.10	439.7	439.73	0.03				
23.20	439.7	440.13	0.43				
23.50	437	440.68	3.68				

Table 10. Differences in Levee Height (feet) Measured from 1995 and 1978 Survey (concluded)

River Mile	*1978*	*1995*	*1995-1978*	*River Mile*	*1978*	*1995*	*1995-1978*
	Keach Drainage LDD				*Hartwell LDD*		
32.70	438.5	440.93	2.43	38.20	438.5	no levee	
33.10	438.5	440.28	1.78	38.70	440	440.03	0.03
33.50	438.5	438.70	0.20	38.90	440	441.30	1.30
34.00	445.0	439.89	-5.11	39.30	440	441.00	1.00
34.30	438.5	440.56	2.06	39.50	440	440.10	0.10
34.70	438.5	438.65	0.15	39.66	440	440.07	0.07
35.20	438.5	438.94	0.44	39.85	440	439.90	-0.10
35.70	438.5	438.57	0.07	40.04	440	440.13	0.13
36.00	438.5	438.53	0.03	40.40	440	441.41	1.41
36.30	438.5	439.15	0.65	40.80	440	440.77	0.77
36.60	438.5	439.15	0.65	41.40	440	440.66	0.66
37.10	438.5	439.29	0.79	41.80	440	440.20	0.20
37.50	438.5	440.99	2.49	42.30	440	440.96	0.96
37.75	438.5	441.17	2.67	42.70	440	441.06	1.06

Table 10 was prepared only for documentation purposes. It records the differences in elevation as a result of the comparisons. Further verification is needed, but this table can be used as a base to identify the locations of greater discrepancies. In addition, the data sets compared were for the lower 41.6 miles only. Cross sections between RM 41.6 and RM 80 were not measured, and the data from RM 80 to RM 159 are not available at this time. Keeping the model current indicative of representative field conditions will require continuous updating of cross-sectional geometries and levee height information.

Summary

This project report is the first in a series of investigations of management options for flood protection in the Lower Illinois River, from Peoria L&D to Grafton. It documents the progress during the first phase of the project, which includes model development, model verification and calibration, and updating DSS files. The "Introduction" discussed the rationale and scope of work for this research. "The Lower Illinois River – Pool 26 UNET Mode" illustrated model coverage, boundary conditions, DSS files, and results of calibration and validation. The DSS update, when data verification is completed, will enable users to examine floods and proposed management strategies using data from the 1970s to the present. However the database verification and model calibration work will be continued in the second phase.

"Frequency Analysis" presents updated information for newly added stations. In order to maintain consistency in the analysis, the same procedures used in the previous analysis were applied. Data covered L&D 25 tail water stage, Cuivre River at Troy, Mel Price L&D pool stage, and Pool 26 pool water stage.

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Illinois State WATER SURVEY

ILLINOIS DEPARTMENT OF NATURAL RESOURCES

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