Implementation and Verification of a One-Dimensional, Unsteady-Flow Model for Spring Brook near Warrenville, Illinois

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Prepared in cooperation with the Illinois Department of Natural Resources, Office of Water Resources and Du Page County, Department of Environmental Concerns

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Implementation and Verification of a One-Dimensional, Unsteady-Flow Model for Spring Brook near Warrenville, Illinois

By MARY J. TURNER, ANTHONY P. PULOKAS, and AUDREY L. ISHII

Prepared in cooperation with the
Illinois Department of Natural Resources, Office of Water Resources and Du Page County, Department of Environmental Concerns

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CONVERSION FACTORS AND VERTICAL DATUM

Multiply	By	To obtain
foot (ft)	0.3048	meter
mile (mi)	1.609	kilometer
square mile (mi^2)	2.590	square kilometer
cubic foot per second (ft^3/s)	0.02832	cubic meter per second

Sea level: In this report, "sea level" refers to the National Geodetic Vertical Datum of 1929 (NGVD of 1929)—a geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada, formerly called Sea Level Datum of 1929.

Implementation and Verification of a One-Dimensional, Unsteady-Flow Model for Spring Brook near Warrenville, Illinois

By Mary J. Turner, Anthony P. Pulokas, *and* Audrey L. Ishii

Abstract

A one-dimensional, unsteady-flow model, Full EQuations (FEQ) model, based on de Saint-Venant equations for dynamic flow in open channels, was calibrated and verified for a 0.75-mile reach of Spring Brook, a tributary to the West Branch Du Page River, near Warrenville in northeastern Illinois. The model was used to simulate streamflow in a small urban stream reach with two short culverts, one with overbank flow around the culvert during high flows. Streamflow data were collected on the reach during three high-flow periods. Data from one period were used to calibrate the model, and data from the other two periods were used to verify the model. Stages and discharges over the periods were simulated, and the results were compared graphically with stage and discharge data collected at 10 sites in the study reach. Errors in simulated stage and discharge were small except when debris, not represented in the model, clogged the culvert.

The effects of changes in physical and computational model parameters also were studied. The model was insensitive to replacement of measured cross sections with interpolated cross sections, especially if the measured thalweg elevation was preserved. Variation of the roughness, slope, and length of the culvert overbank section, as well as the chosen representative measured cross section, caused only slight changes in the simulated peak stage and discharge. Changes in the modeled culvert area caused large differences in the simulated high flows in the vicinity of the culvert, whereas simulated low flows were unaffected. At all flows, the misrepresentation of the culvert area caused the simulated water-surface elevations to deviate from the measured elevations, especially on the falling limb of the stage hydrograph. The FEQ model, including the routines for modeling culvert and overbank flows, was evaluated as accurate and effective for this application.

INTRODUCTION

Many stream basins in the suburban Chicago county of Du Page are undergoing rapid urbanization that causes changes in the delineation of the regulatory flood plain. The streams in the area tend to have low slopes with large storage potential. Traditional steady-flow methods may incorrectly describe the flood-plain hydraulics because only the effects of changes in conveyance are considered and the changes in streamflow routing caused by changes in storage are neglected in steady-flow methods. Unsteady-flow models are tools that can be used by flood-plain managers to aid in the determination of or changes in the regulatory flood plain because the models take into account storage effects.

The unsteady-flow model used in this study, Full EQuations (FEQ) dynamic-wave model is based on the de Saint-Venant equations (de Saint-Venant, 1871) and is unique in that many control structures and stream features can be simulated. These structures and features include weirs, bridges, culverts, overbank areas, embankments, and several dynamic controls, such as pumps and dams. They can be represented in the model by function tables computed in the companion program, Full EQuations UTiLity (FEQUTL) (D.D. Franz, Linsley, Kraeger Assoc., Ltd.; and C.S. Melching, U.S. Geological Survey, written commun., 1995) and accessed as needed during streamflow simulation. In another report, Ishii and Turner (in press) describe the calibration and verification of an FEQ model of a 30.6-mi reach of a large stream system, the Fox River, in Illinois, containing controlling features, such as bridges, low-head dams, and flat slopes, during a dam-controlled, unsteady-flow period. The data collection for this verification study was planned to test the applicability of the culvert and overbank-flow routines during natural flood periods on a small stream reach (0.75 mi), Spring Brook, a tributary to the West Branch Du Page River, in Illinois.

The study was done in cooperation with the Illinois Department of Natural Resources, Office of Water Resources, and Du Page County, Department of Environmental Concerns. Other phases of the study include the documentation of FEQ and its companion program, FEQUTL, the verification of the FEQ model for use on a large stream system (Ishii and Turner, in press) and the collection of data used for the large stream-system verification (Turner, 1994).

Purpose and Scope

The purpose of this report is to document the implementation, calibration, and verification of a one-dimensional, unsteady-flow model of a relatively small stream with culverts and overbank flow, Spring Brook, a tributary to the West Branch Du Page River, in Illinois, and to verify that the results of the unsteady-flow simulation, applying typical assumptions of culvert and overbank representations, are appropriate. The ability to reproduce a period of unsteady floodflow with the calibrated model is demonstrated by comparing the simulation results of two periods to stage and discharge data for those periods. The sensitivity of the model to computational and physical parameters was tested by varying the parameters. Model results that demonstrate a sensitivity to parameter changes or that are specific to culvert or overbank applications are shown graphically.

Study Area

Spring Brook originates in Wheaton, Ill., and flows through unincorporated areas of Du Page County to the West Branch Du Page River at river mile 93.45. The basin has moderate slope (25–200 ft/mi) and covers 8.14 mi^2. The total length of Spring Brook is 6.8 mi. The reach discussed in this report is 0.75 mi long and begins in Roy C. Blackwell Forest Preserve. The reach flows on the border of the preserve, through a wooded residential area, and through the property of Cenacle Retreat House ending at the mouth of the stream at the West Branch Du Page River.

Data were collected on the reach at two major areas and at the upstream most and downstream most points of the reach. A streamflow-gaging station was located at the upstream boundary, Spring Brook at Forest Preserve near Warrenville, Ill., hereafter referred to as the Forest Preserve (site 1) (fig. 1, fig. 2, table 1). Data from this site served as the upstream boundary condition for the Spring Brook model.

A street, named Morris Court, crosses Spring Brook at about river mile 0.4. This area is the location of the partial-record streamflow-gaging station Spring Brook at Morris Court at Warrenville, Ill. (fig. 1, fig. 3, table 1). The area is referred to as Morris Court in the report and includes Morris Court headwater (site 2) and Morris Court tailwater (site 3). Also, the culvert through which the stream passes at this area is referred to as Morris Court culvert.

The stream passes through and ends on the property of Cenacle Retreat House where the streamflow-gaging station, Spring Brook above West Branch Du Page River near Warrenville, Ill., is located (fig. 1, fig. 4, table 1). The area is referred to as Cenacle and includes six sites: Cenacle total flow (site 4), Cenacle headwater (site 5), Intermediate staff gage (site 6), Cenacle tailwater (site 7), Departure-overbank junction (site 8), and Overbank (site 9). Also, the culvert through which the stream passes at this area is referred to as Cenacle culvert.

A downstream boundary streamflow-gaging station, Spring Brook at Warrenville, Ill., was located at the mouth of Spring Brook about 20 ft upstream from the West Branch Du Page River. This site will be referred to as Mouth (site 10).

The average slope for the 0.75-mi reach is 10.5 ft/mi (slope of 0.0020) (fig. 5). The study reach was selected because the flow at Cenacle culvert is known to overflow the bank often and the overbank

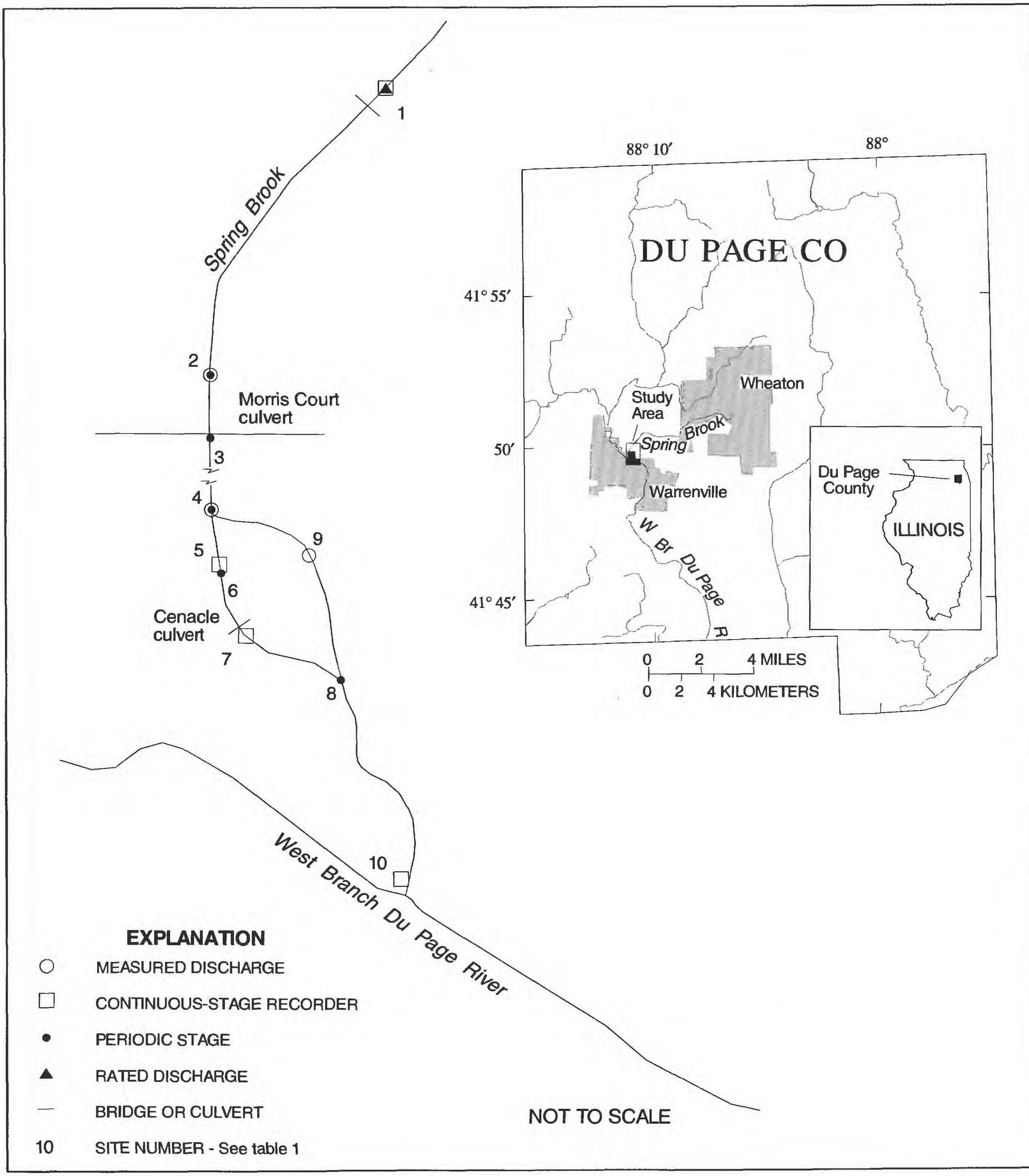


Figure 1. Location of Spring Brook study reach and data-collection sites.

Figure 2. Spring Brook at Forest Preserve near Warrenville, Ill.

Figure 3. Spring Brook at Morris Court at Warrenville, Ill.

Table 1. Data-collection sites for the Spring Brook, tributary to the West Branch Du Page River, study reach in Illinois
[ft, feet]

Site number (fig. 1)	River miles above downstream end of study reach	U.S. Geological Survey station name and downstream order number	Description	Short name	Type of data collected
1	0.750	Spring Brook at Forest Preserve near Warrenville 05540091	Continuous and crest-stage gage at foot bridge	Forest Preserve	Continuous stage, rated discharge
2	.410	Spring Brook at Morris Court at Warrenville 05540092	Staff and crest-stage gage 75 ft upstream from Morris Court	Morris Court headwater	Periodic stage, measured discharge
3	.392	....do.	Reference point on downstream right wingwall of Morris Court	Morris Court tailwater	Periodic stage
4	.275	Spring Brook above West Branch Du Page River near Warrenville 05540093	Cenacle total discharge measurement section 95 ft upstream from culvert	Cenacle total flow	Periodic stage, measured discharge
5	.265	....do.	Continuous-stage gage 40 ft upstream from culvert	Cenacle headwater	Continuous stage
6	.260	....do.	Staff gage 10 ft upstream from culvert	Intermediate staff gage	Periodic stage, inferred discharge[1]
7	.251	....do.	Continuous-stage gage on downstream face of culvert	Cenacle tailwater	Continuous stage
8	.244	....do.	Reference point 40 ft downstream from culvert	Departure-overbank junction	Periodic stage
9	.233	....do.	Swale discharge measurement section on left bank of culvert	Overbank	Measured discharge
10	.005	Spring Brook at Warrenville 05540094	Continuous-stage gage at confluence with the West Branch Du Page River	Mouth	Continuous stage

[1]Discharge was not measured directly but is inferred by subtracting the measured overbank flow from the measured total flow.

Figure 4. Spring Brook above West Branch Du Page River near Warrenville, Ill.

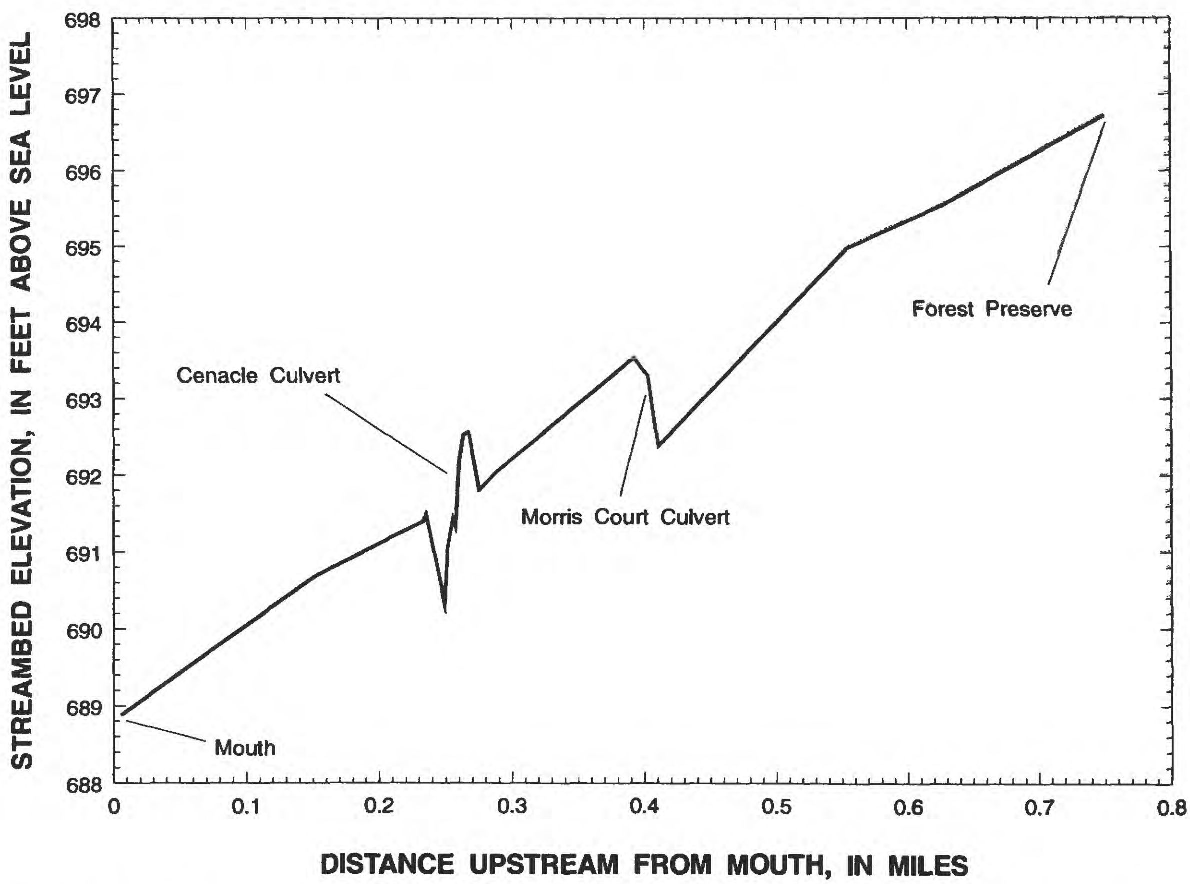


Figure 5. Study-reach bottom elevation and data-collection sites, Spring Brook, a tributary to the West Branch Du Page River, in Illinois.

flow is confined in the overbank channel so that it is easily measured.

The mean annual flow at the Forest Preserve (site 1), with a drainage area of 6.83 mi^2, for October 1991–September 1993 was 15.2 ft^3/s. The instantaneous peak flow for the period of record was 194 ft^3/s on March 23, 1993. The minimum daily mean flow was 4.1 ft^3/s on July 30, 1993 (Zuehls and others, 1994).

The topography of the Spring Brook watershed is relatively flat with a relatively uniform gradient. The drainage basin narrows near the mouth so that lateral inflow for the study reach was negligible. The present topography was formed by glaciation and modified by natural water-erosion processes, farming, and urbanization (G. Nicholas Textor, Envirodyne Engineers, written commun., 1989). Further development continues to modify the drainage of the basin.

Acknowledgments

The authors are grateful to Du Page County, Department of Environmental Concerns, particularly Hsueh-Ju Sherrie Chang and Christopher Vonnahme, for providing initial model data. Also, the cooperation and assistance of personnel of Du Page County Forest Preserve and Cenacle Retreat House, throughout the data-collection process, are appreciated.

DATA COLLECTION

Data for this study were collected during periods of high floodflow for use in testing the capabilities of FEQ. The study reach was selected because it includes two culverts, one of which is known to constrict the flow so that overbank conditions result, with flow bypassing the culvert and reentering the main channel downstream. The overbank flow bypasses the culvert through a swale separate from the main channel, where it can be measured easily (see fig. 6). The sites on Spring Brook and the type of data collected at each are listed in table 1 and are shown on figure 1.

Continuous-stage recorders were installed at four sites in the study reach (see fig. 1). At the upstream boundary, a continuous-stage recorder was installed at the Forest Preserve (site 1), and a stage-discharge rating was developed for that site. Two continuous-stage recorders were installed at Cenacle culvert—one upstream and one downstream from the culvert structure. The headwater gage (Cenacle headwater, site 5) was installed approximately 40 ft

Figure 6. Spring Brook above West Branch Du Page River near Warrenville, Ill., during overbank flow.

upstream from the culvert entrance. The tailwater gage (Cenacle tailwater, site 7) was installed on the downstream face of the culvert (see fig. 1). The fourth continuous-stage recorder was installed at the Mouth (site 10) of Spring Brook to provide downstream boundary-condition data. Continuous-stage and discharge data were collected for this study during September 1991–August 1993.

The main interest of the study was to simulate the streamflow at Cenacle culvert because of the constricting culvert and overbank flow during high flow. Therefore, flow measurements were taken intensively at and near the culvert. In addition to the continuous-stage recorders, a staff gage was installed approximately 10 ft upstream from the culvert inlet (Intermediate staff gage, site 6). This staff gage more closely approximated the culvert approach depth and, were it feasible, the headwater continuous-record gage would have been installed there rather than further upstream. The Cenacle tailwater (site 7) continuous-record gage was installed at the outlet of the culvert to verify the type of flow through the culvert and provide stage data in case the cross section at the outlet would become the control on the flow through the culvert. However, high flow through the culvert was always supercritical near the inlet and through the length of the culvert, so the Cenacle tailwater (site 7) stage was always affected from upstream. The flow remained supercritical for about 8 ft downstream from the culvert outlet and formed standing waves for approximately 20 ft further downstream. The microfeatures of the culvert causing these flows were not accounted for in the model representation of the stream reach, and it would be impractical to add more detail to the model representation. Consequently, no comparable model output for the Cenacle tailwater (site 7) data is available. The flow in the channel returned to subcritical by the next data-collection point at the Departure-overbank junction (site 8).

Continuous-stage data for the selected flood periods at Cenacle headwater (site 5) and tailwater (site 7) are shown in figure 7. The fall in water-surface elevation through the culvert is large during the flood peaks, whereas the channel constriction caused by the culvert is effectively much less during low flow. At low flow, the stage-discharge relation for the Cenacle tailwater (site 7) is controlled from downstream. A small riffle about 20 ft downstream from the culvert was the control at low flow. The downstream riffle did not appear to affect the stage-discharge relation at the Cenacle headwater (site 5) gage.

At Morris Court, crest-stage and staff gages were installed approximately 75 ft upstream from the culvert (Morris Court headwater, site 2), and a reference point was placed on the right downstream wingwall (Morris Court tailwater, site 3). A drainage ditch on the left bank, about 20 ft upstream from Morris Court, was observed during high flow; it provided only minimal storage with no measurable inflow.

During high flow (fig. 6), crews made current-meter discharge measurements of the total streamflow (Cenacle total flow, site 4) and of the overbank flow (Overbank, site 9) as well as measurements of the water-surface elevation, primarily at Cenacle but also at the Forest Preserve (site 1) and Morris Court (sites 2 and 3). During the highest flows, a boat was needed to make the discharge measurements of the Cenacle total flow (site 4); the Overbank (site 9) was always wadable. Discharge through the culvert was determined by subtracting the Overbank (site 9) discharge measurement from the total streamflow discharge measurement (site 4). Three high-flow periods were measured: December 30, 1992–January 6, 1993; March 23–24, 1993; and June 7–9, 1993.

On June 7–9, 1993, crews made two Cenacle total flow (site 4) discharge measurements, three Overbank (site 9) discharge measurements, and frequent stage measurements. During high flow, wading discharge measurements were impossible, and a boat was not available in time to measure the higher flows. The period of June 7–9 was selected for the model calibration for two reasons: (1) The culvert was not blocked, and (2) fewer measurements were available to verify during this period.

During the December 1992–January 1993 flood, 11 discharge measurements were made at Cenacle total flow (site 4). Crews were at the site December 30–31 and January 4–7. The culvert was clear on December 30–31 but was clogged with debris on January 4–5. Six discharge measurements were made while the culvert was clogged with debris. On January 6 at 1135 hours, the debris was cleared, and three more discharge measurements were made. The peak measured discharge at Cenacle total flow (site 4) in the stream was 178 ft^3/s, with a peak measured discharge at Overbank (site 9) of 108 ft^3/s on January 4.

On March 23–24, 1993, five measurements were made, all while the culvert was clear except

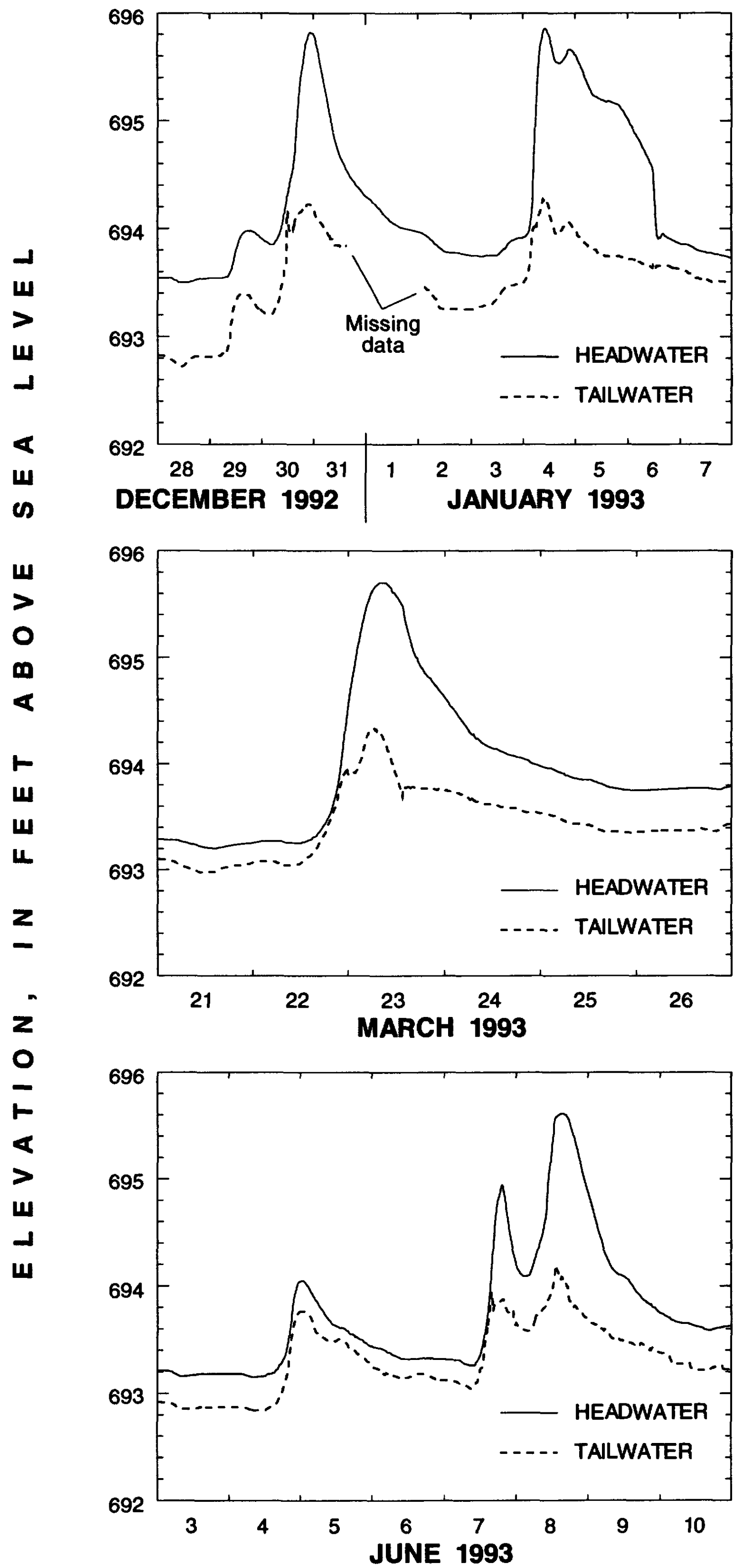


Figure 7. The headwater and tailwater continuous-stage data at Cenacle culvert (sites 5 and 7) on Spring Brook, a tributary to the West Branch Du Page River, in Illinois.

for the first measurement. The measured discharge at Cenacle total flow (site 4) ranged from 48 to 134 ft^3/s with Overbank (site 9) flows from 0 to 71 ft^3/s. The December–January and the March streamflow data were used for verification of the model.

DESCRIPTION OF A ONE-DIMENSIONAL, UNSTEADY-FLOW MODEL

The one-dimensional, unsteady-flow, numerical model used for this study is based on the integral form of the equations of continuity (conservation of mass) and motion (conservation of momentum). The equations are valid under the assumptions described by Cunge and others (1980, p. 8). Unsteady, open-channel flow is described in the equations by two dependent variables—discharge or velocity and stage, depth, or cross-sectional area. These variables are dependent on distance and time. By assuming the dependent variables are continuous, differentiable functions, the differential equations of unsteady flow may be approximated as finite-difference equations for discrete intervals of time and distance (Lee, 1989, p. 6). At discontinuous points where the equations do not apply, internal boundary conditions are used. The nodes of the internal boundary conditions are supplied with relations, computed in FEQUTL routines, of the flows and water-surface elevations. The routines are based on algorithms developed in Dalrymple and Benson (1967) for overbank flow and Bodhaine (1968) for culvert flow. (D.D. Franz, Linsley, Kraeger Assoc., Ltd.; and C.S. Melching, U.S. Geological Survey, written commun., 1995)

IMPLEMENTATION AND CALIBRATION OF THE SPRING BROOK MODEL

Implementation of a one-dimensional, unsteady flow model for Spring Brook was completed by inputting the channel geometry, including field selected values of Manning's roughness coefficient (n), and hydraulic structure data into FEQUTL. Function tables were computed in FEQUTL using the cross-section data (fig. 8). In FEQ, the study reach of the Spring Brook channel was modeled as a network of eight branches, each with two exterior nodes. Tributary and lateral inflows were insignificant along the study reach so that no tributary branches were modeled. Two culverts are represented in the model. At the second culvert, an overbank channel is modeled. The model schematic with the model output and data locations are shown in figure 9.

The model was initially calibrated with the June 1993 flood data using the technique of Ishii and Wilder (1993). The technique is to fit two simulations—one with discharge upstream and stage downstream boundary conditions (discharge-stage, Q–Z) and the other with stage upstream and downstream boundary conditions (stage-stage, Z–Z)—by adjusting Manning's n. The best fit Manning's n was then the starting point for the traditional calibration technique of fitting the discharge-stage boundary-condition simulation to the data. The best fit with the Ishii and Wilder calibration technique also was the best fit with the traditional calibration technique (fig. 10). The stage-stage boundary condition was used only for this calibration, and the discharge-stage boundary condition was used for all other simulations.

Channel Geometry

Channel geometry for Spring Brook is represented by 17 cross sections. Most cross-sectional data were obtained from Du Page County Department of Environmental Concerns. All surveys are referenced to the same bench mark, which was linked to the first-order level net of the NGVD of 1929. Significant errors in the bed-slope representation are unlikely because the reach is represented by numerous cross sections. Selected cross sections used for modeling are shown in figure 8. Some of the cross sections have been truncated so that the same horizontal and vertical scales are shown in all plots.

The channel is about 30 ft wide at the upstream boundary, gradually narrows to about 15 ft wide at the upstream side of the culvert at Cenacle Retreat House, then gradually returns to a 30-ft width before entering the West Branch Du Page River. The channel bed slope is about 10.5 ft/mi (.0020) for the reach (fig. 5). When the depth at Cenacle total flow (site 4) reaches about 2.8 ft, overbank flow begins to bypass the culvert. The overbank channel begins about 93 ft upstream from the culvert entrance on the left bank. At about 35 ft upstream from the culvert entrance, high ground separates the two channels, and overbank flow bypasses the culvert by flowing over a cut-grass swale. The flow reenters the main channel approximately 116 ft downstream from the culvert exit.

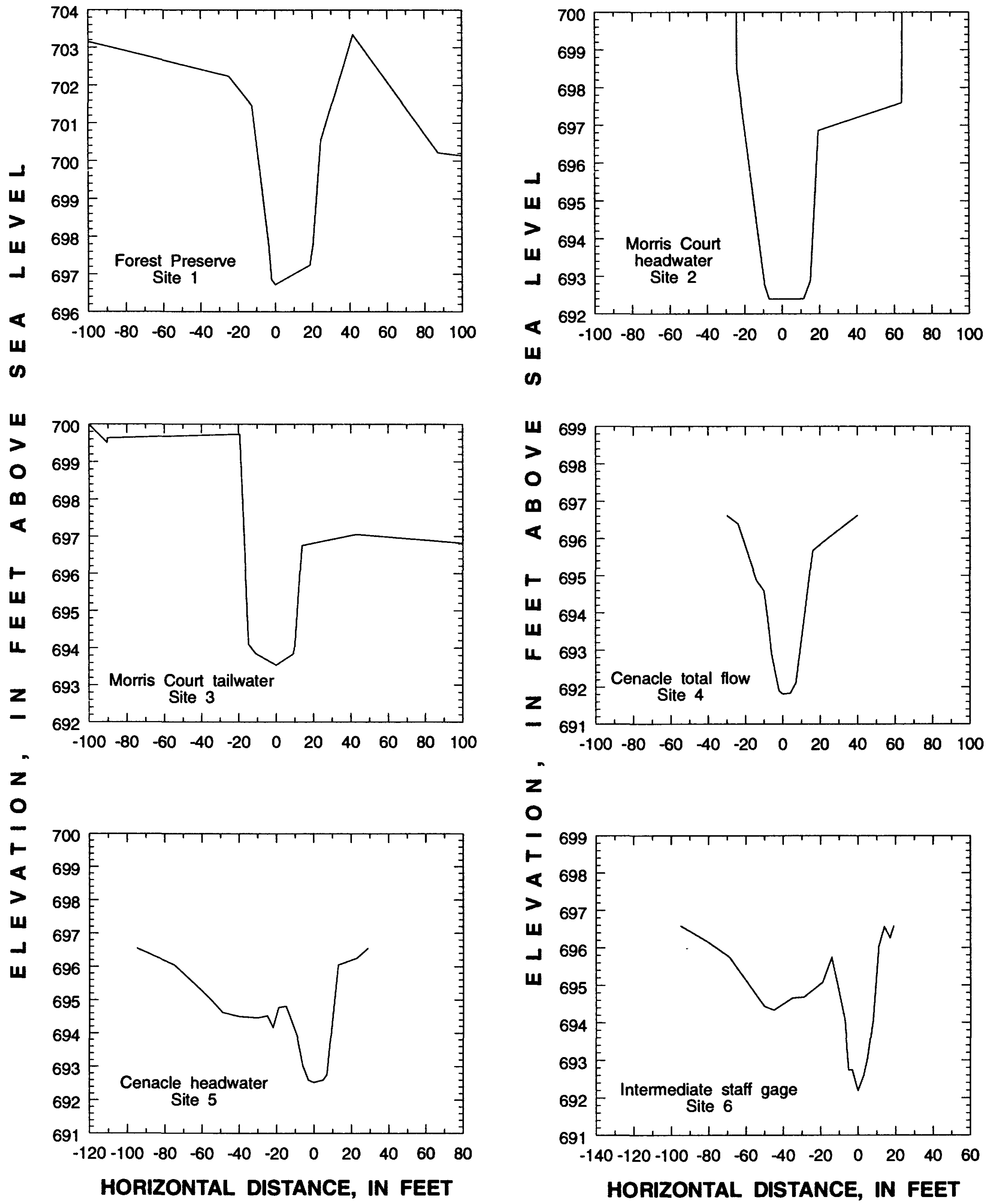


Figure 8. Plots of the nearest surveyed cross section to discharge and stage data-collection sites for the study reach on Spring Brook, a tributary to the West Branch Du Page River, in Illinois.

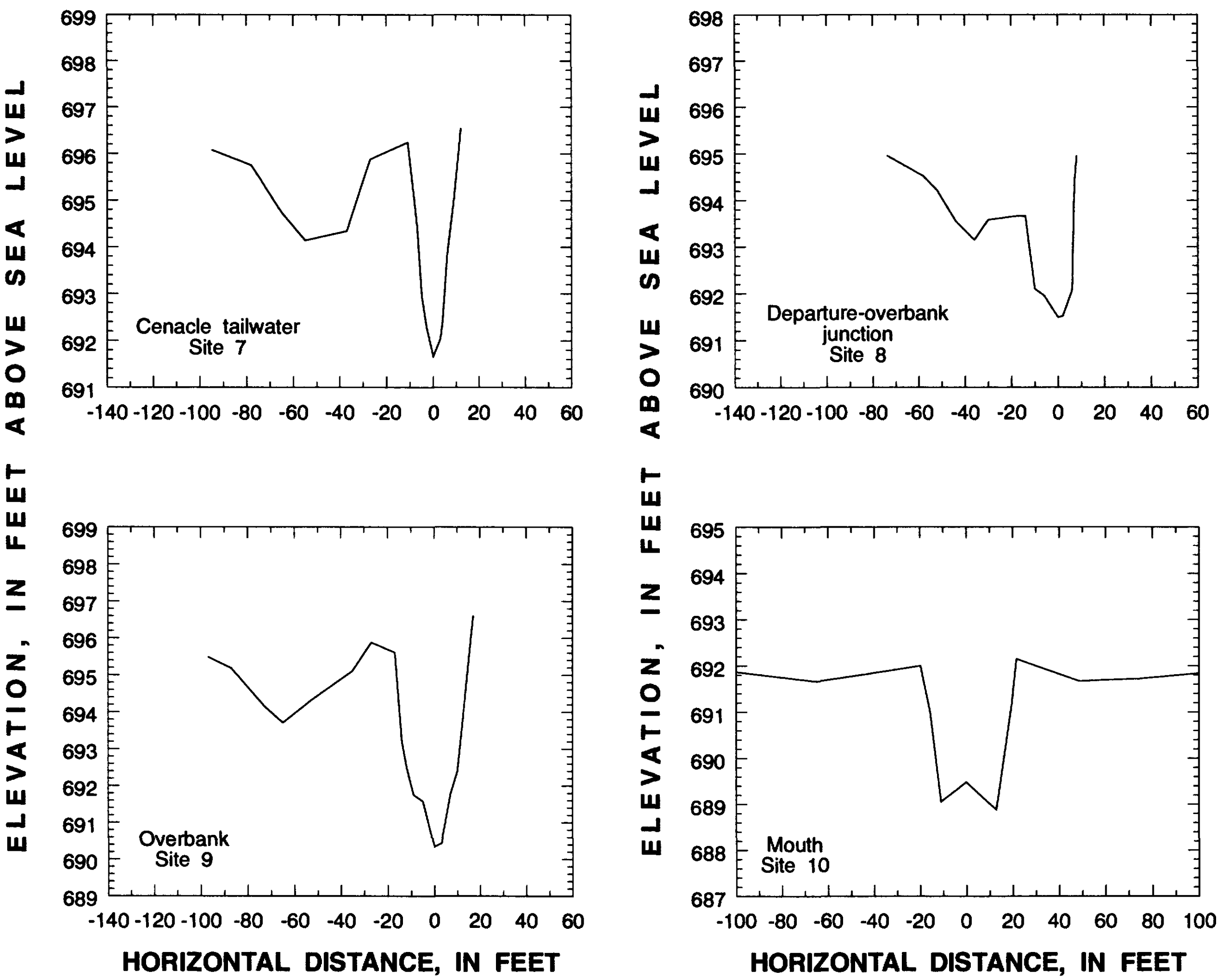


Figure 8. Continued.

Culverts

A channel constriction by a culvert results in rapidly varied flow where acceleration, rather than boundary friction, has the most effect in defining the flow (Bodhaine, 1968). Therefore, the de Saint-Venant equations for gradually varying flow cannot be applied. In FEQ modeling, a culvert structure is simulated with the FEQUTL routine, CULVERT. A two-dimensional table of the flow through the culvert as a function of the water-surface elevations at the approach and departure sections is developed in CULVERT using a steady-flow energy conservation equation. The flow over the associated roadway is included in the function table. The table is accessed in FEQ simulation when the culvert is encountered. Standard culverts and culverts with slight deviations from standards can be represented in CULVERT. Bodhaine (1968) classifies culvert flow into six types for peak-flow estimation. The six peak-flow types classified by Bodhaine, as well as nine additional flow types to describe transitional flows, are recognized in CULVERT. (D.D. Franz, Linsley, Kraeger Assoc., Ltd.; and C.S. Melching, U.S. Geological Survey, written commun., 1995).

Dimensions of the culvert, as well as nearby cross sections of the channel, are entered for use in computation of the flow through the culvert and over the associated roadway. The cross sections used for routing through a culvert are (1) the approach section, located at least one culvert opening width upstream from the culvert entrance, (2) the culvert entrance, (3) the culvert exit, (4) the beginning of the departure reach, and (5) the end section of the departure reach, located where the velocity of the stream is no longer affected by the culvert.

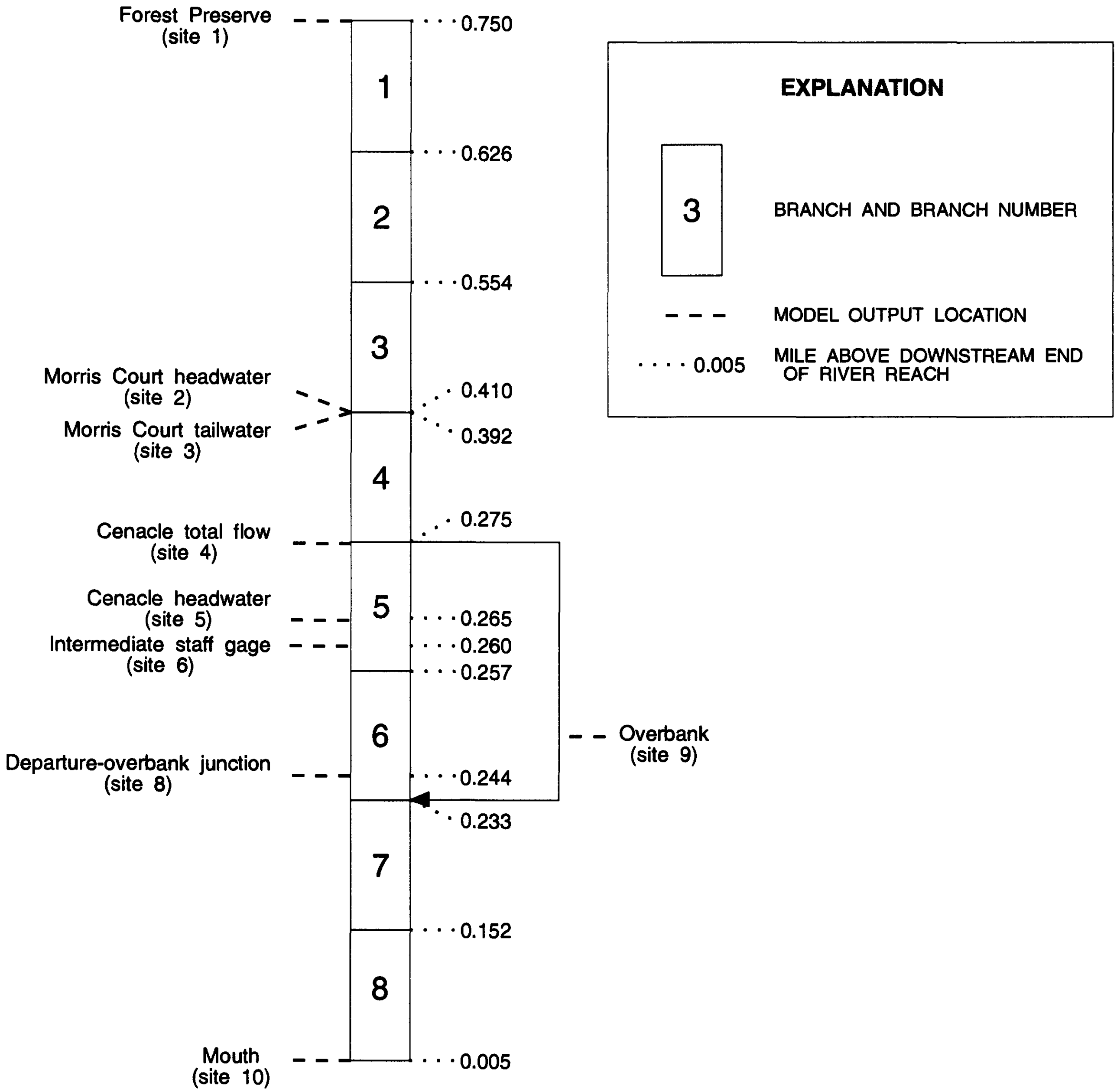


Figure 9. Schematic of the FEQ model of Spring Brook, tributary to the West Branch Du Page River, in Illinois, showing output locations.

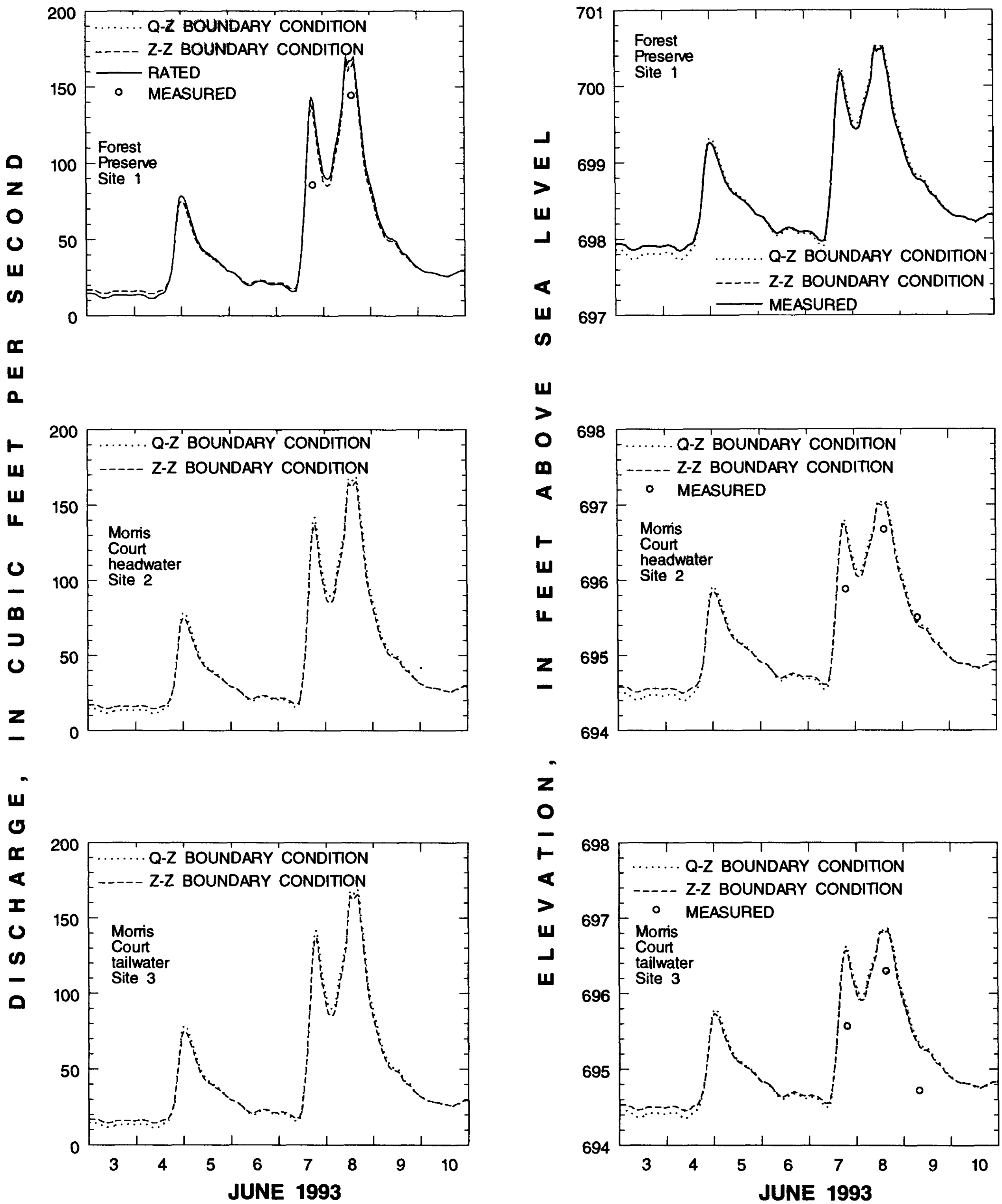


Figure 10. Measured or rated and simulated discharge and stage for the June 1993 calibration period on Spring Brook, a tributary to the West Branch Du Page River, in Illinois.

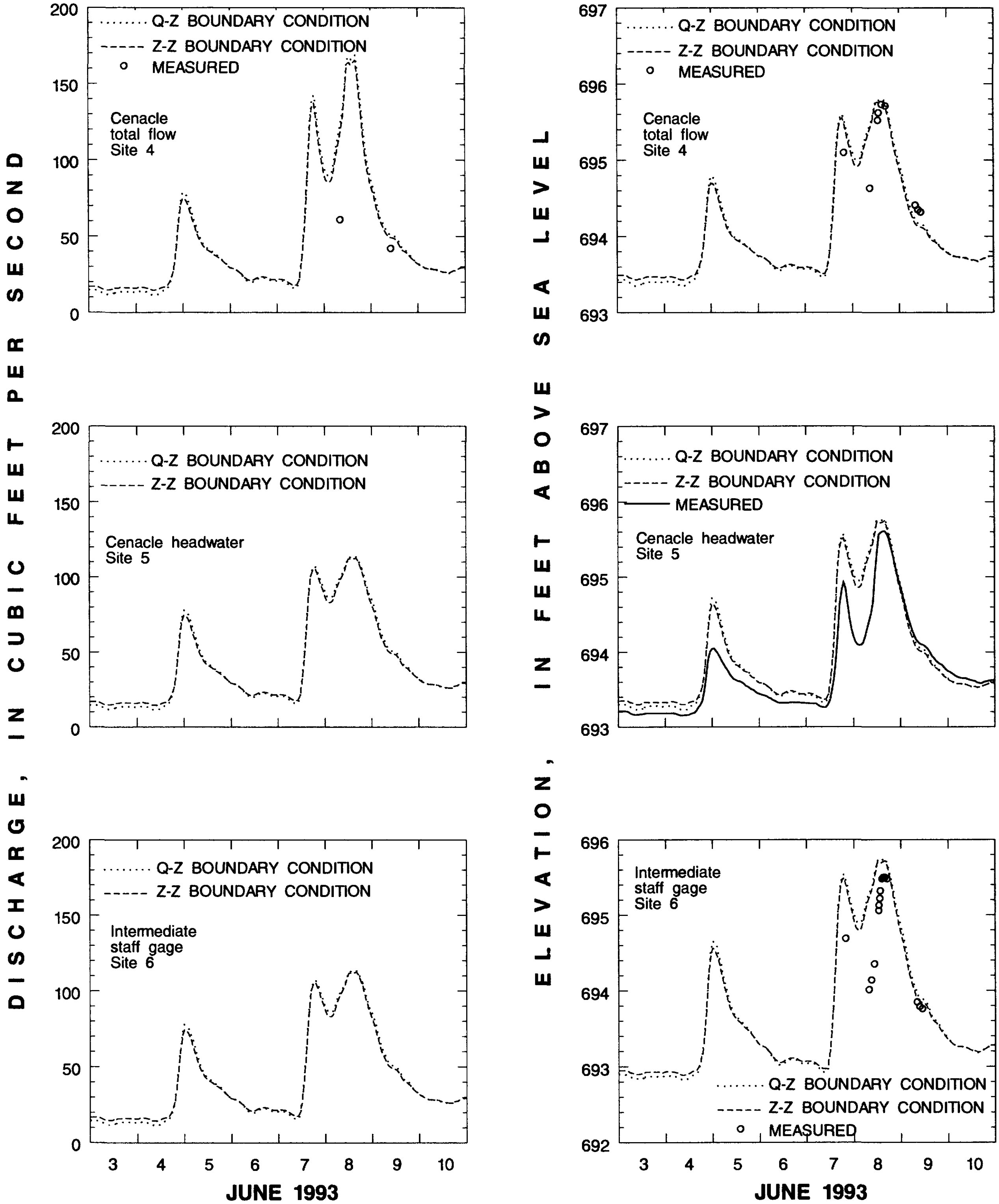


Figure 10. Continued.

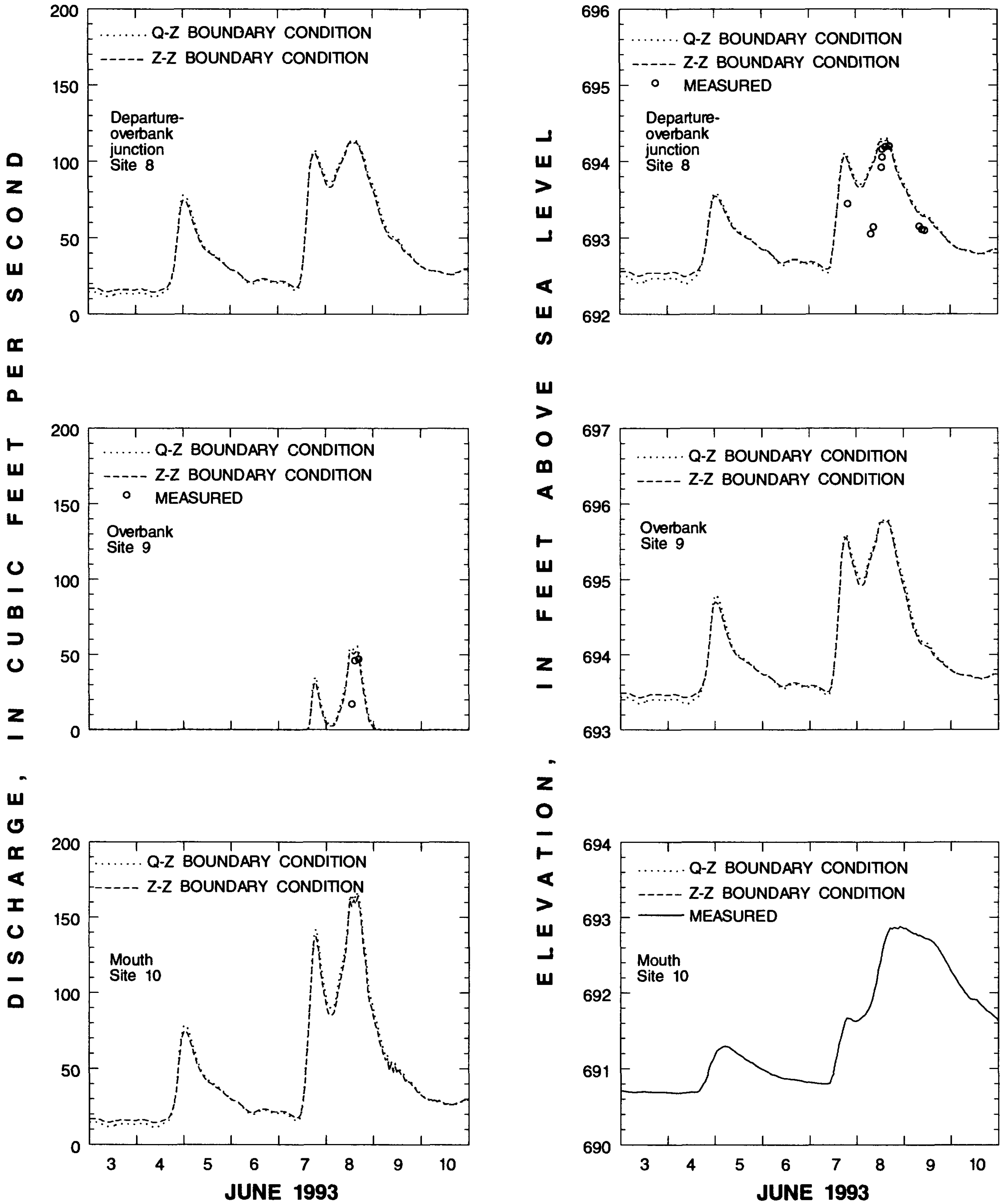


Figure 10. Continued.

Two culverts are present in the study reach—one at Morris Court and one at Cenacle Retreat House. Morris Court culvert is actually a crude bridge over Spring Brook on Morris Court and provides access to two homes on the left bank of the stream (fig. 3). The culvert is maintained by the home owners, and replaced a former structure. The concrete supports of the former structure have dropped into the stream. The structure was modeled as a box culvert, approximately 27 ft wide by 5 ft high. Because the length of the structure is only 12 ft, the length to depth ratio is less than 3, and the equations of culvert flow may not apply at this site. Overall, the structure is not standard and was not closely studied as an example of a CULVERT application. When the Morris Court culvert was removed from the model in sensitivity analysis, the change in routing was very small.

Cenacle culvert is a 4-ft by 6-ft corrugated-metal pipe arch culvert 16.2 ft in length. The slope of the Cenacle culvert is adverse, rising about 0.1 ft from the entrance to the exit (fig. 4). The culvert was built with 135 degree wingwalls on the upstream side. The left wingwall is 5.6 ft long and the right wingwall is 1.9 ft long; concrete and stone debris have been deposited in front of the wingwalls, rendering them ineffective. The wingwalls are not represented in the model. A rock-riffle control, approximately 10 ft upstream from the culvert, is not modeled because it affects only low flow. No flow over the pathway at Cenacle culvert was observed.

The approach section at Cenacle culvert was surveyed 9 ft upstream from the culvert. The departure ending section was surveyed 20 ft downstream from the culvert exit, and the section for the beginning of the departure reach was surveyed 6 ft below the exit. The approach cross section and departure ending cross section also are in the FEQ input, appearing at the branch ends, which are just upstream and downstream from the culvert.

Overbank Flow

During high-flow periods, multiple flow paths may be present at a structure. Each of the paths, including through, over, and around a structure, must be modeled individually in FEQ to accurately estimate the losses of each flow path. Flow around a structure in the flood plain of a stream is simulated in the CHANRAT routine in FEQUTL. A two-dimensional table for a short prismatic channel is computed in CHANRAT, assuming one direction of flow and subcritical flow at all flow levels. The bottom slope, the channel length, and a representative cross section of the channel are required in CHANRAT. Flow and steady-flow water-surface profile through the channel for each of a series of upstream heads with a range of downstream heads is computed in the CHANRAT command in FEQUTL with the equation

$$\frac{dx}{dy} = \frac{1-F^2}{S_o - S_f},$$

where

- x is the distance along the channel, increasing downstream;
- y is the maximum depth of flow in the channel;
- F is the Froude number (Q/Q_c);
- S_o is the bottom slope;
- S_f is the friction slope (Q^2/K^2), where Q is the flow rate; Q_c is the critical flow rate; and K is the conveyance.

Overbank flow around the culvert at Cenacle Retreat House is modeled with CHANRAT. Between 90 and 35 ft upstream from the culvert entrance, when the depth reaches approximately 2.8 ft at Cenacle total flow (site 4), part of the flow begins to leave the main channel and flow through a swale on the left bank of Spring Brook (fig. 6). The flow reenters the main channel approximately 116 ft downstream from the culvert exit. The starting and ending points of the overbank channel were estimated from measured cross sections. From these points, the slope of the overflow channel was calculated at 0.0069, and the length was about 220 ft. During the measured high-flow periods, the Overbank (site 9) flow measurements ranged from 4 to 110 ft^3/s with respective Cenacle total flow (site 4) measurements from 93 to 180 ft^3/s.

The overbank area of a cross section 30 ft downstream from the culvert exit was chosen to represent the prismatic channel in CHANRAT. A modeled vertical frictionless wall prevented water from occupying the left overbank in cross sections representing the main channel near Cenacle. Otherwise, the overbank channel would be incorrectly represented twice in FEQ—once in the CHANRAT function table and once in the overbank area in the main cross sections.

Boundary and Initial Conditions

Boundary and initial conditions were obtained from data collected at the sites on Spring Brook. The upstream boundary for the model was rated discharge at the Forest Preserve (site 1). The discharge was computed from a stage-discharge relation developed for that site. Hysteresis was not apparent in the rating curve. The downstream boundary was the stage at the mouth (Mouth, site 10) of Spring Brook where the stream enters the West Branch Du Page River. For calibration, an additional simulation was run with stage at the Forest Preserve (site 1) for the upstream boundary and stage at the Mouth (site 10) as the downstream boundary. No lateral inflow was simulated in the reach. Initial conditions were estimated from stage and discharge records on Spring Brook. Similarly to Ishii and Turner (in press), any errors in estimating initial conditions were corrected in the simulation within 24 hours of the starting time.

Roughness Coefficients

Initial estimates of the roughness coefficient (Manning's n) were made in a field reconnaissance of the study reach. Estimating techniques from Chow (1959), Arcement and Schneider (1989), and Barnes (1967) were used. With the data from the June 1993 storm, the values of Manning's n were calibrated from the initial estimates. Calibration was done by comparing discharge and stage results from simulations with two different sets of boundary conditions: one with stage data for both upstream and downstream boundaries and one with discharge for the upstream boundary and stage for the downstream boundary. To calibrate the model, the roughness was varied in appropriate cross sections until the plotted model results from the two sets of boundary conditions nearly matched. Once the calibration was established, further attempts were made to fit the discharge-stage simulation alone to the measurement data, but the attempts did not result in overall improvements in the calibration. Thus, the technique of fitting simulations of different boundary conditions appeared to be a suitable means of calibration. Results of the calibrated simulation are shown in figure 10.

Calibrated values of Manning's n were highest in the upstream most reaches, ranging from 0.065 to 0.068, most likely the result of brush and fallen timber in the stream. In the reaches just above and below Cenacle culvert, Manning's n ranged from 0.045 to 0.060, reflecting the presence of large rocks lining the streambed. Between the culvert area and the mouth at the West Branch Du Page River, Manning's n was calibrated to a lower value of 0.035, representing a muddy stream bottom and less brush and debris.

The overbank area near Morris Court was covered with dense brush. Manning's n in this area was set to 0.070. The banks of Spring Brook around Cenacle are muddy with some gravel and large rocks. Manning's n in this reach was calibrated at a value of 0.035. The overbank area near Cenacle Retreat House was a cut-grass lawn. The value of n in this area was set to 0.040. Manning's n was varied in the prismatic representation of the channel with flow bypassing Cenacle culvert, until the balance of simulated flow between the total channel and the overbank matched field measurements. A value of 0.045 was selected.

Calibration Results

For calibration purposes, a simulation was run with continuous-stage readings at both the upstream and downstream boundaries. However, in general practice, simulation is usually done with discharge at the upstream boundary. Thus, discussion of the calibrated results refers to the discharge-stage boundary-condition simulation as compared with the data only. Calibration of Manning's n did result in highly similar simulation results under both sets of boundary conditions (fig. 10) during the June storm.

A problem was apparent at the upstream boundary, the Forest Preserve (site 1). The discharge measurements made on June 7–8 do not fall on the stage-discharge relation for the site. Note that as a boundary condition, the discharge was forced to equal the rated discharge record at this location. Therefore, errors here would propagate throughout the simulation. At Morris Court headwater (fig. 10, site 2), the simulated peak on June 8 is about 0.3 ft higher than the measurement, while the simulated peak on June 7 is 0.8 ft higher than the measurement. The differences are greater on the tailwater side of Morris Court (site 3), where the simulated peak on June 8 is about 0.5 ft higher than the measurement, while the simulated peak on June 7 is 1.0 ft high.

Near Cenacle culvert (fig. 10, sites 4–9), the simulated stage and discharge results closely fit the measurements at the peak late in the day on June 8. Also, the division of flow between the main channel and the Overbank (site 9) was calibrated to a very close fit (fig. 10, site 9). However, for sites 4 through 8, the smaller peaks on June 5 and 7 and the wave trough on the early morning of June 8 do not fit well. On June 5, the simulated stage at the peak is high by 0.6 ft at Cenacle headwater (site 5), whereas the simulated stage at the wave trough on June 8 is high by 0.8 ft. Water-surface-elevation measurements at the Departure-overbank junction (site 8) show a similar pattern. The discharge measurement at the Cenacle total flow (site 4) seems to show that the discharge also has been simulated high. This discharge measurement is extremely low compared with the rated discharge at the Forest Preserve (site 1), is uncharacteristic of the flow routing observed throughout the study, and has no known physical explanation. The consistent pattern in the results and the discharge measurements indicate that the continuous-rated discharge boundary condition at the Forest Preserve (site 1) is in error during the June storm. Although few measurements were taken at the Morris Court sites, the measurements fit a similar pattern of error. Possible explanations for the problem are that the stage readings at the Forest Preserve were inaccurate due to a malfunctioning potentiometer or partial blockage inside the gage. Also, the actual stage-discharge relation at the gage might have been temporarily changed due to debris buildup just downstream.

Although the simulation results for part of the traditional calibration period are inaccurate in some locations, the calibration of Manning's n was effective, since the Ishii and Wilder (1993) calibration technique only involved fitting the discharge-stage boundary-condition simulation to the stage-stage boundary-condition simulation. Attempts to fit the discharge-stage boundary-condition simulation to the data were unsuccessful, which is most likely because the roughness was already well calibrated. The persistent lack of fit was a result of boundary-condition data error that could not be compensated by changing roughness. Apparently, the problem at the gage at the Forest Preserve (site 1) did not occur during the two verification periods. Furthermore, the calibration seems to be independent of this error, as the simulated results for the verification periods are accurate (see "Verification of the Spring Brook Model" section).

VERIFICATION OF THE SPRING BROOK MODEL

Model verification was accomplished by comparing results from the calibrated model with data collected during two periods of unsteady flood-flow, December 1992–January 1993 and March 1993. Stages and discharges were simulated using discharge for the upstream boundary and stage for the downstream boundary.

Hydraulic Routing Results

The hydraulic routing for the upstream reach (Forest Preserve and Morris Court headwater and tailwater (table 1, fig. 1; sites 1, 2, and 3, respectively)), will be discussed in this section and the remainder of the sites in the reach will be discussed in the "Culvert and Overbank Results" section. This upstream reach includes Morris Court culvert, but eliminating the culvert table for Morris Court had little effect on the simulation results. Few data for the upstream sites are available except for the upstream boundary condition of the Forest Preserve (site 1). Two discharge measurements were made at Morris Court during the December–January high flows; both matched the simulation within 8 percent (fig. 11, site 2). The simulated stage compared well to measured data during both verification periods at Morris Court (figs. 11 and 12, site 2). Minimal storage in the drainage ditch upstream from Morris Court was not represented in the model. If the storage were represented in the model, simulated results would possibly better match the measured discharges. Presently, all simulated discharges are higher than measured discharges.

Culvert and Overbank Results

The simulation of culvert flow is complicated by the large number of features that can change the control on the stage-discharge relations. Important features include the culvert size, shape, slope, and roughness; inlet and outlet geometry; and capacities of the upstream and downstream channels (Morris and Wiggert, 1972, p. 283). It is not always possible to

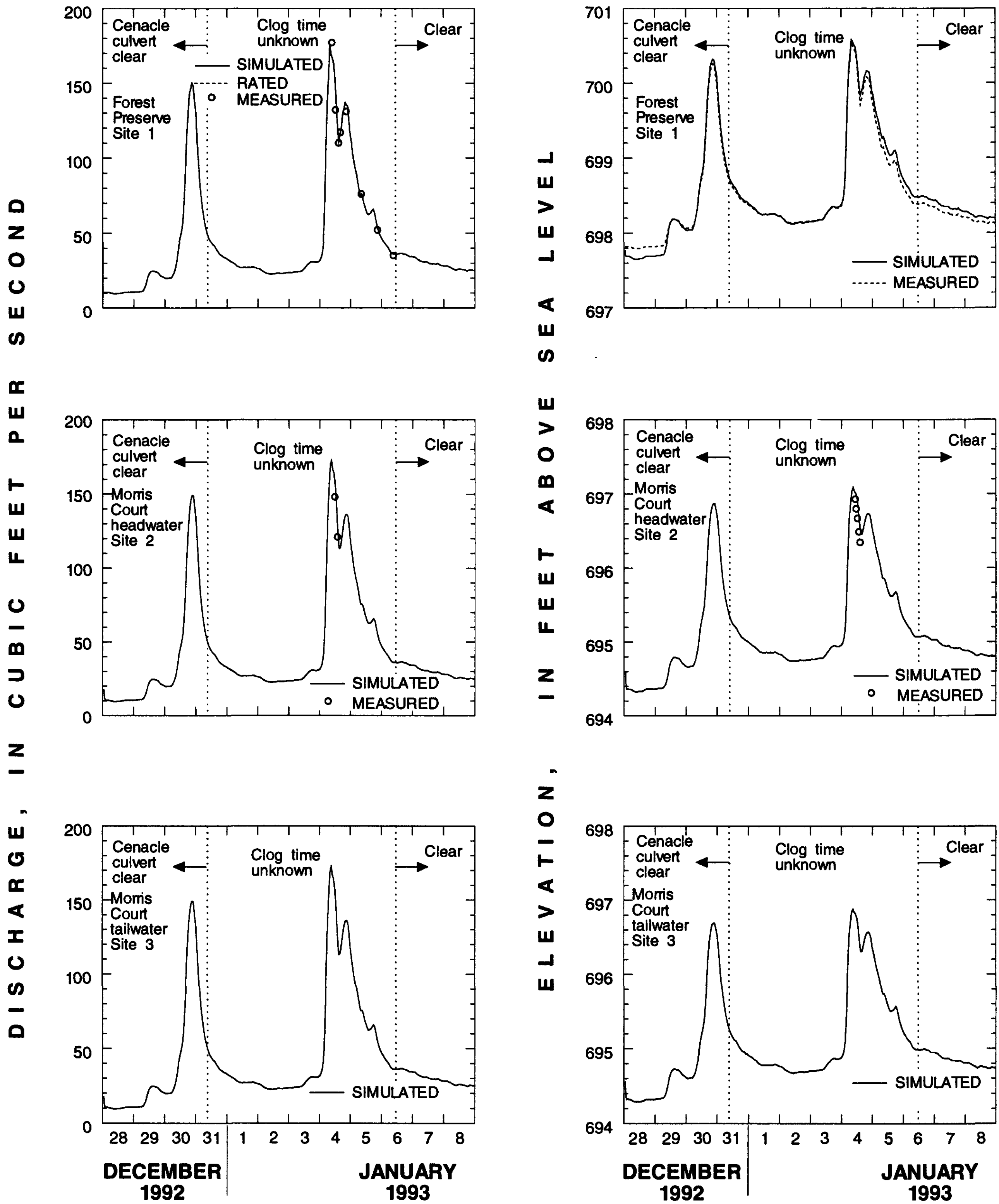
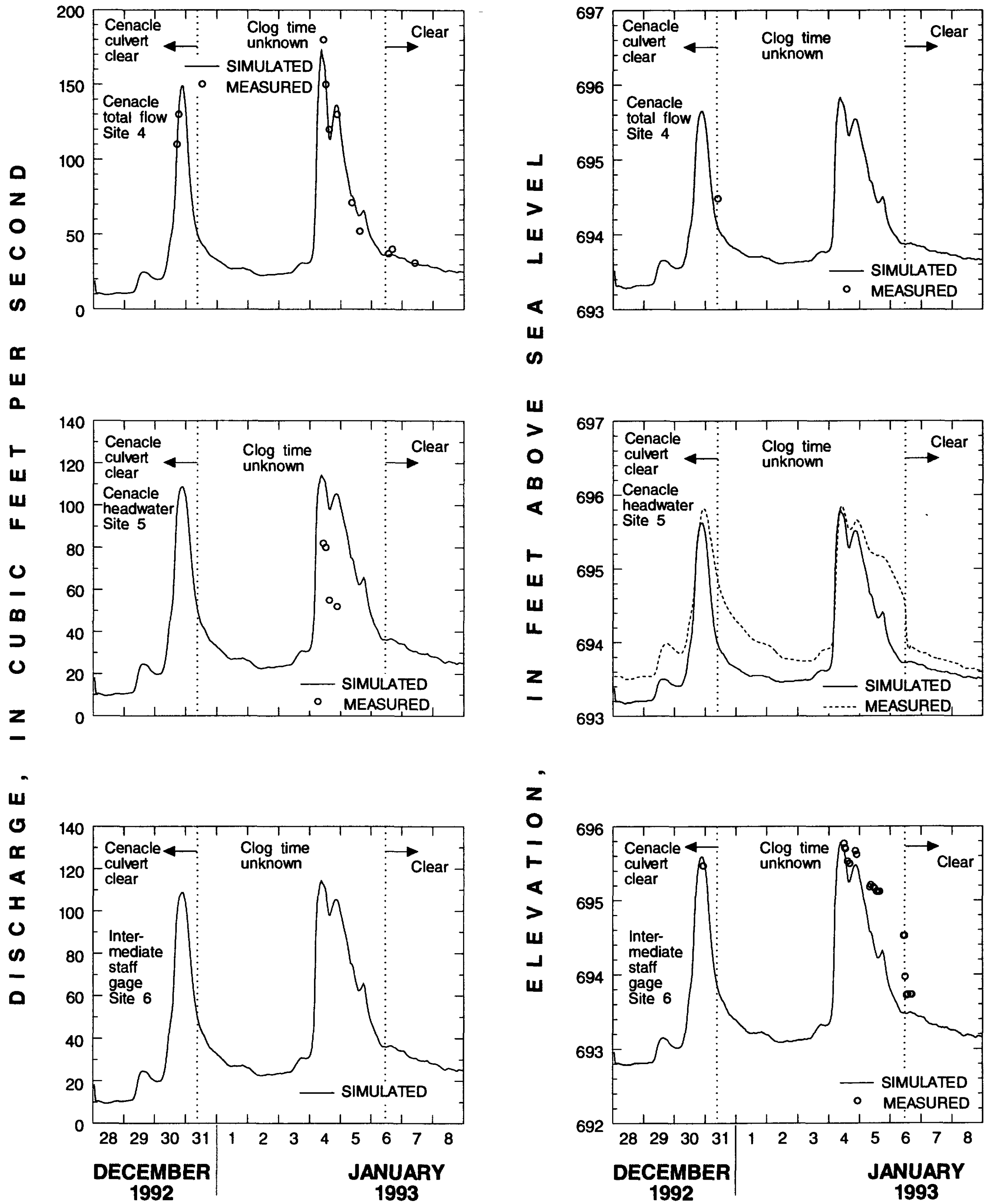


Figure 11. Measured or rated and simulated discharge and stage for the December 1992–January 1993 verification period on Spring Brook, a tributary to the West Branch Du Page River, in Illinois.



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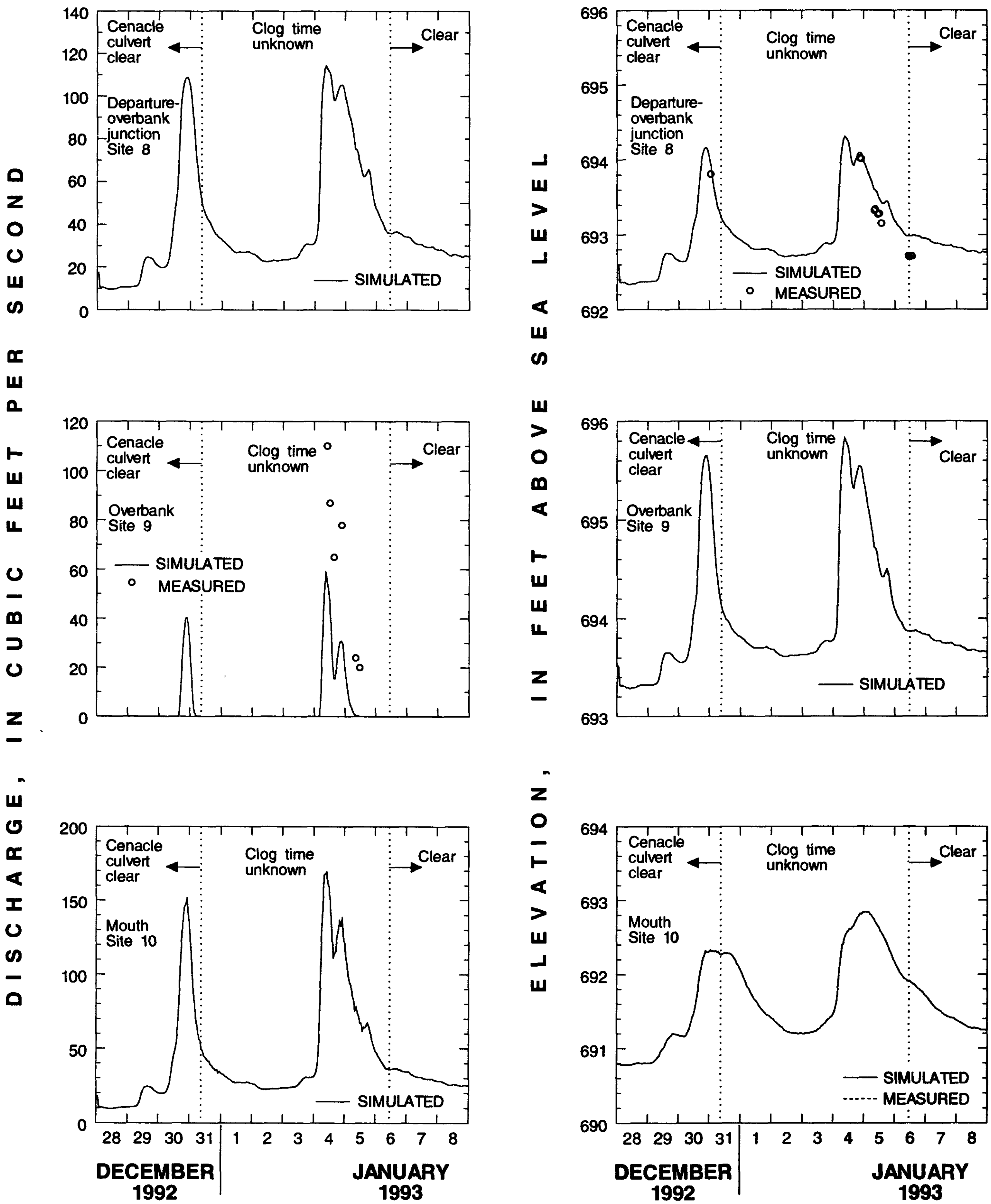


Figure 11. Continued.

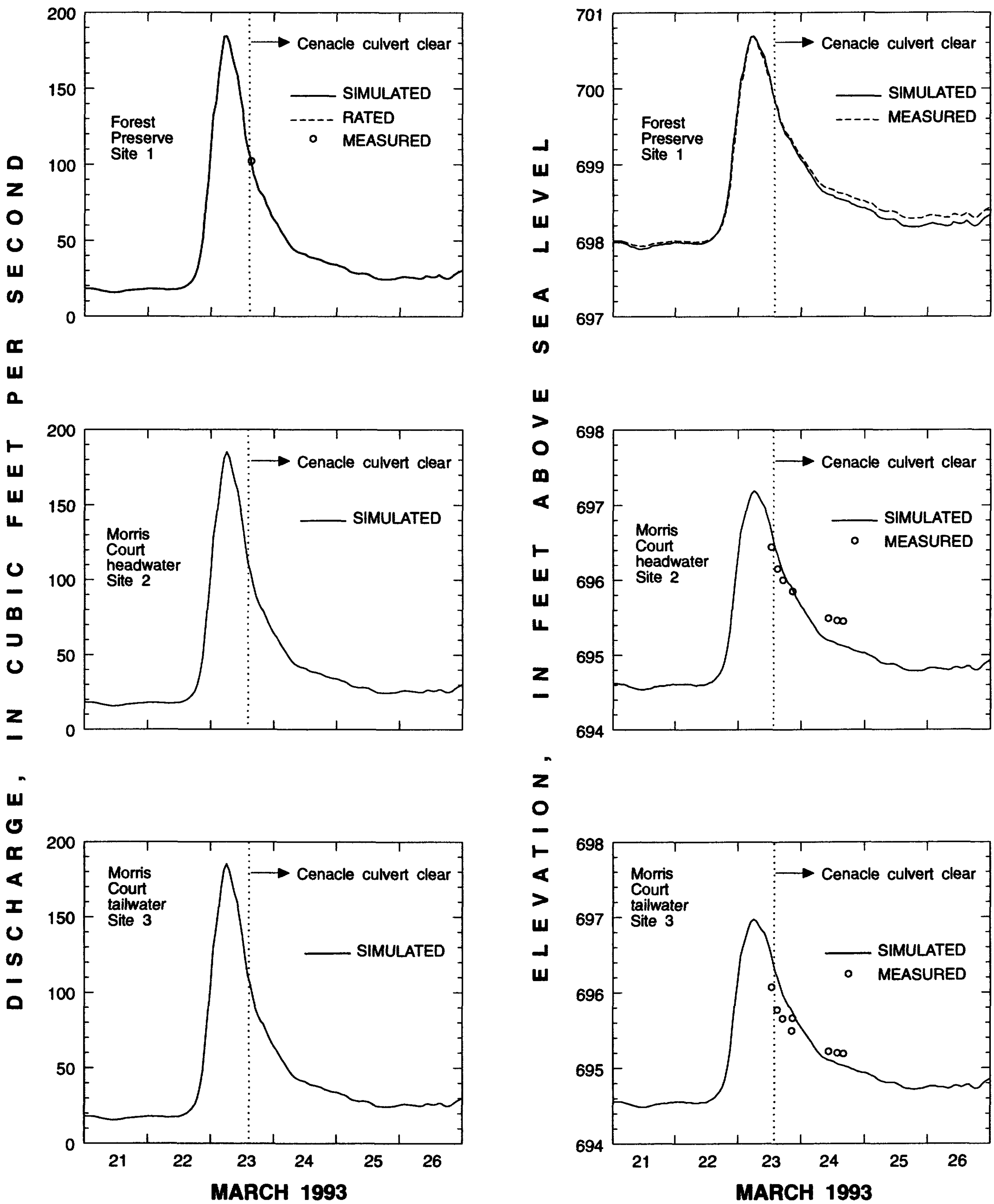


Figure 12. Measured or rated and simulated discharge and stage for the March 1993 verification period on Spring Brook, a tributary to the West Branch Du Page River, in Illinois.

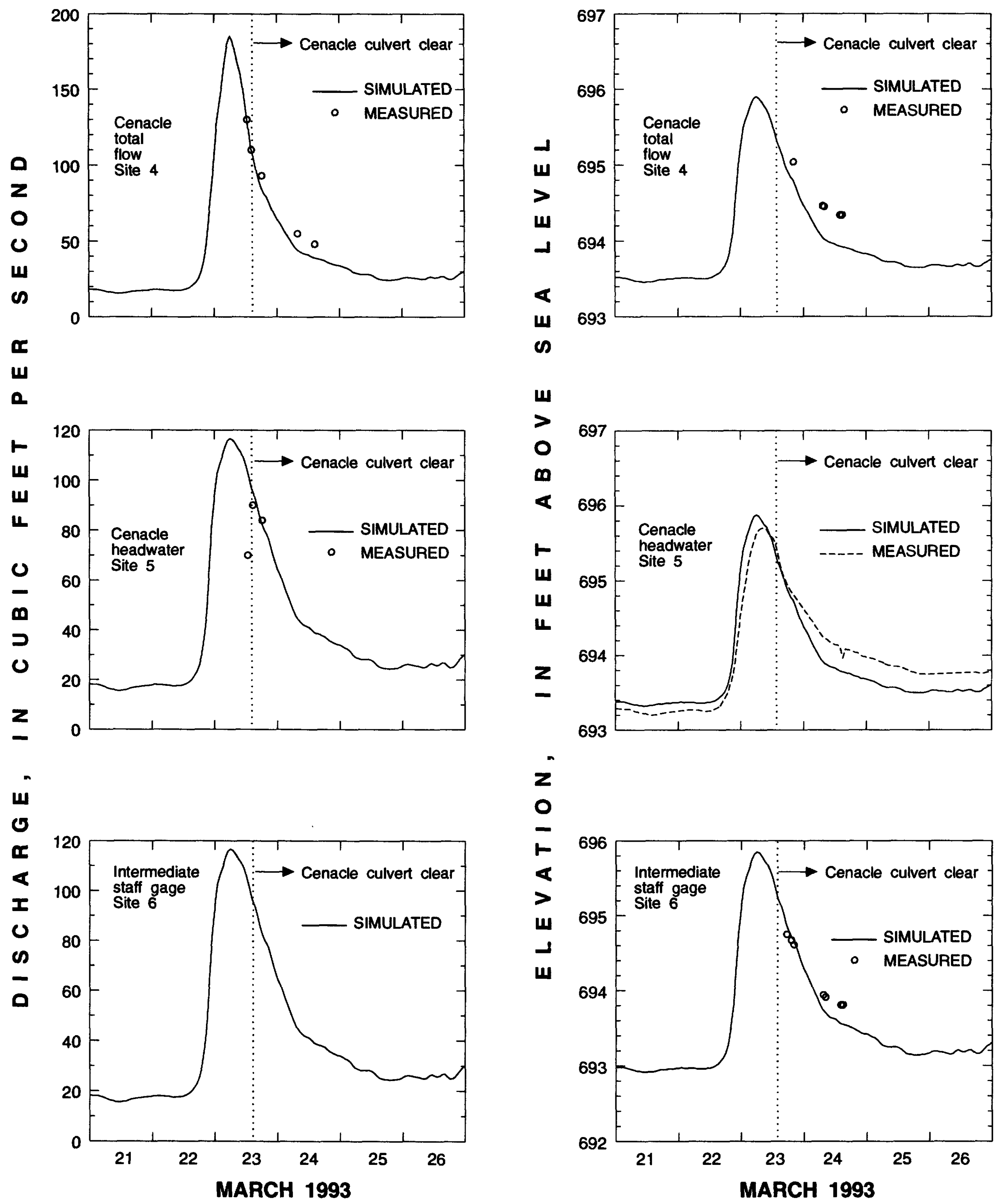


Figure 12. Continued.

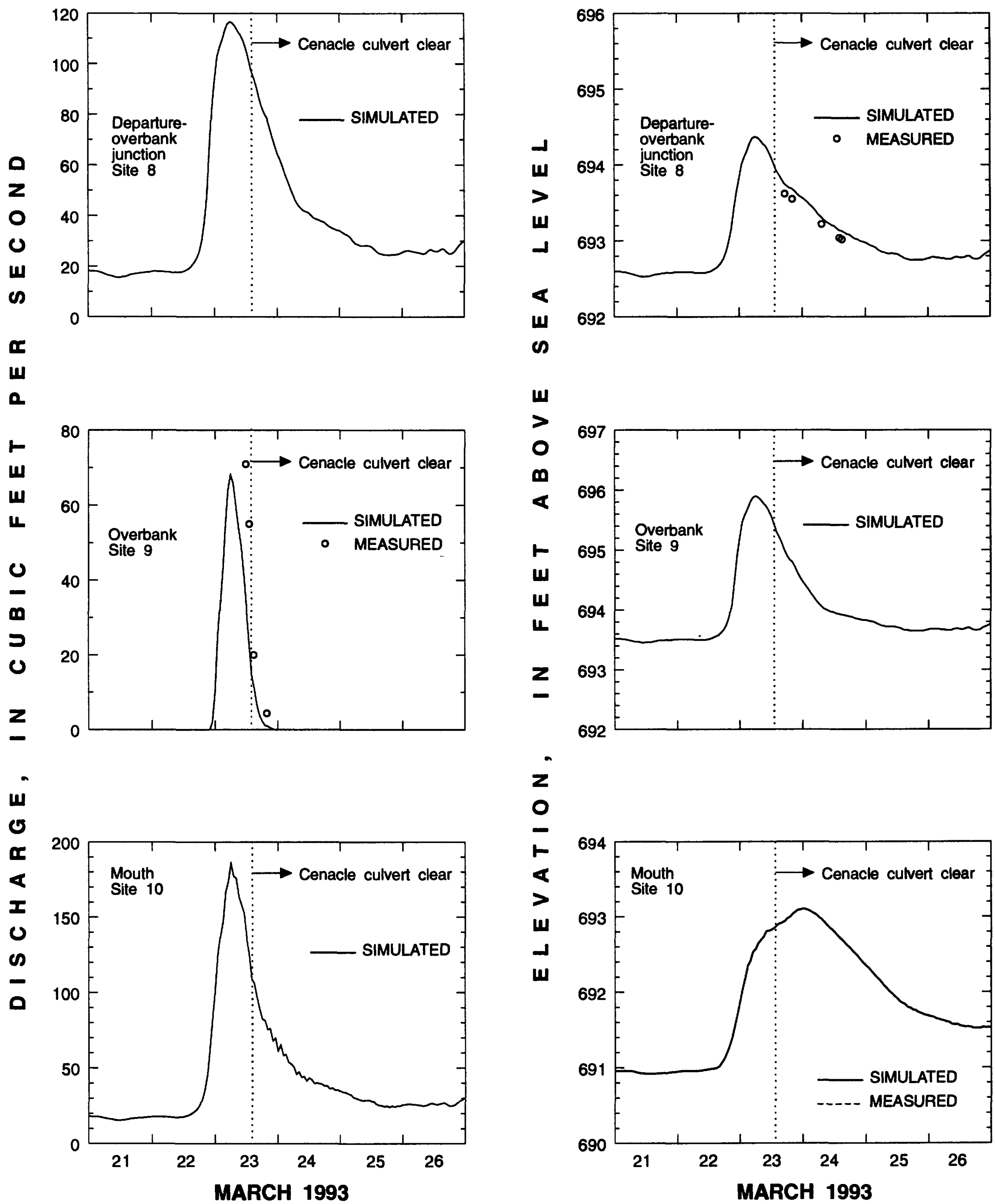


Figure 12. Continued.

determine from a field inspection at low flow which culvert features will affect the discharge at high flows. Culvert flow has been divided into six types of common flow conditions, depending on the location of the stage-discharge controls, based on the results from several theoretical and empirical studies (Bodhaine, 1968). An additional nine types of flow conditions are recognized in the CULVERT routine. Because of the limited culvert conditions that have been studied and the complexity of culvert flow, extrapolation to culvert conditions in the field is approximate, and evidence other than heuristic rules must be considered in determining what flow types are present during high flow.

This study was intended to provide an evaluation of an application of FEQ in practice; therefore, no attempt was made to adjust the model input data to obtain the flow types observed in the field, except in the selection of the variable that determines the upper limit of Type I and Type II flow. At low flow, the culvert did not cause an appreciable contraction in the flow, and the flow was subcritical throughout the length of the culvert. This is classified as Type III flow in Bodhaine (1968) and was the type of flow that was simulated in this study for low flows.

At high flow, prior to submergence of the culvert inlet, the flow was observed in the field to be supercritical at both the culvert outlet and inlet and, therefore, was assumed to be supercritical throughout the length of the culvert. Because the flow was supercritical at the inlet, the flow-type classification should be Type I. However, Bodhaine (1968) indicates that Type I flow is applicable only when the culvert barrel slope is supercritical. Because the Cenacle culvert slope was adverse, Type I flow was not computed in FEQUTL. The reason for the observed supercritical flow at the inlet was probably the contraction caused by the converging sides of the pipe-arch shape as the water surface rose. The supercritical-flow state was maintained despite the adverse slope of the culvert because the culvert was short enough so that the head loss due to friction was very small. Thus, the flow through the culvert resembled flow over a broad-crested weir (Brater and King, 1976, p. 5–23).

The lack of backwater effect and the dominant effect of critical flow were correctly assumed. Until the inlet was submerged, the simulation utilized the equation for Type II flow. Both Type I and Type II flows are computed from the equations for critical depth (Bodhaine, 1968, p. 3). A difference between the two flow types is the assumed location of critical flow. The computational difference for the flow types is small. Computation of discharge for Type II flow does not include an elevation term, which would be a negative value for the adversely sloping culvert, and additional losses within the culvert are accounted for in Type II flow. Additional losses would be small due to the relatively short length (16.2 ft) of Cenacle culvert (Bodhaine, 1968). Another difference would be the difference in the velocity heads of the approach section, which is small in most cases and considered negligible in this study.

When the water-surface elevation at the culvert entrance exceeded 1.1 times the height of the culvert, the upper limit for Type I and Type II flows declared in the input is reached and the tables generated in FEQUTL are computed for a transition to Type V flow. The transition is accomplished by using the equations for Type V flow, with the discharge coefficient adjusted to make the flow transition smoothly from Type II flow. The water-surface elevation in the departure section was never near submerging the culvert crown at the outlet; thus, no transition to Type IV flow was required in FEQ simulation, although tables including Type IV conditions were computed in FEQUTL.

The comparison of the simulated and measured flows and water-surface elevations near Cenacle culvert was complicated by microfeatures that are not represented in the model. At low flows of less than 40 ft^3/s, boulders and cobbles in the channel bottom of approximately 1 ft or less in size created riffles about 20 ft upstream and downstream from the culvert. These riffles controlled the stage-discharge relations at low flows but were not included in the model because their effect was completely drowned out at the higher flows at which the culvert controlled the stage-discharge relation. The riffles would have created a slight difference in the water-surface elevation and storage upstream. However, natural variation in the channel geometry accounted for by the selection of cross-section locations apparently was sufficient to account for the headwater stage, even at low flow.

The results for the verification simulations are shown in figures 11 and 12. The data collected for the December 1992–January 1993 floods (fig. 11) demonstrate the effect transitory features, not represented in the model, may have on discharge and stage. In this case, a debris jam at the inlet of the culvert created additional storage upstream from the culvert, raised the water-surface elevation, and caused more flow around the culvert. It was not possible to estimate the time at which the debris jam effectively blocked part of the culvert inlet area or how much of the culvert was blocked, but it is apparent that the blockage caused more flow in the overbank and less flow in the culvert than was simulated. The total flow was accurately simulated, both in amount and timing (fig. 11, site 4). The debris jam was pushed through the culvert at 1135 hours on January 6, 1993, resulting in rapidly falling stage upstream from the culvert.

The effect of the debris jam is not apparent at Cenacle total flow (site 4) with the possible exception of slightly higher than simulated flows during the recession of January 6 and 7. More discharge was measured than was simulated, possibly because discharge was leaving storage. At the Cenacle headwater (fig. 11, site 5) the effect of the debris jam on both the measured discharge and the stage is apparent. The measured discharge through the culvert was as low as 50 percent of the simulated discharge. The water-surface elevations were not greatly affected at the peak elevations but did show considerable effects during the flow recessions, especially the recession of January 5 and 6, when the simulated elevation was as much as 1.0 ft lower than the measured elevation. Elevations would not be expected to differ as much as discharge in this case because the overbank flow path requires the same elevations at its ends as the model branches that connect to the culvert require at their ends.

As expected, the relation of the simulated and measured flows for the overbank section was opposite of the culvert flow relations. The measured flows exceeded the simulated flows, as the debris jam kept the water-surface elevation high enough to send more discharge into the overbank and around the culvert. As the debris jam is not represented in the simulation, the discharge in the overbank section was underestimated by the same amount that discharge through the culvert was overestimated. Simulated total flow was accurate.

Simulated peak-flow water-surface elevations at the Departure-overbank junction (site 8) were close to the measured elevations with an average absolute error of 0.14 ft for the three measurements made on December 31, 1992 and January 4–5, 1993. The measurements made during the flow recession were slightly lower than simulated values, with an average error of 0.17 ft for the six measurements. The elevations may have been affected by the extra storage provided behind the debris jam. Evidence for this is noted in the slightly higher than simulated elevations at Cenacle total flow (site 4) and Intermediate staff gage (site 6) upstream from the debris-blocked culvert.

The data collected during the March verification period matched the simulated results more closely than either the December–January verification data or the June calibration data (figs. 10–11). In March, the culvert was partially blocked with debris during the first two overbank and first culvert measurements. The effects of the blockage are clearly evident at Cenacle headwater and Overbank (fig. 12, sites 5 and 9, respectively). Similarly to the December–January flood, the total flow measured at Cenacle total flow (site 4) is accurately simulated. The two measurements made after the blockage was re noved at the culvert also are close to the simulated discharge for the culvert. Despite the blockage, which strongly affected the discharge through the culvert, the simulated elevation for the Cenacle headwater (site 5) was only slightly higher and peaked slightly earlier than the recorded elevation data. The effect of the blockage on the discharge-elevation relation is most evident on the recession limb of the flood hydrograph for the December–January flood since the blockage was cleared before the March flood recession was completed and there was little effect on the stage at that time.

SENSITIVITY ANALYSIS OF SPRING BROOK MODEL

Sensitivity analyses are performed to determine how simulation results are affected by variations in input parameters. Parameter variations most likely to affect the results include variation in channel geometry, the boundary and initial conditions, and the roughness coefficient (Ishii and Turner, in press). The boundary and initial conditions were tested similarly to Ishii and Turner (in press), with similar results. Adjustments to boundary datums were important only when stage was used for both boundaries, and adjustments in initial conditions were damped out within 24 hours of the simulation. Computational-convergence parameters, such as time step, distance step, iterations per time step, temporal integration weighting factor, and convergence criterion, were tested to ensure that the discrete solutions to the flow equations approach the exact solution and that the model had converged. For the verification, the maximum time step was 1 hour, the maximum number of iterations was set to 5, the temporal integration weighting factor was 0.6, and the convergence criteria was 0.10. These parameters, as well as the distance step, also were varied similarly to the sensitivity analysis in Ishii and Turner (in press) with similar results.

The simulation and data from the March period were used as the base simulation in the sensitivity tests because the hydrograph was well-defined and single-peaked for that period. Also, the data set from that period was complete and debris in Cenacle culvert was not present during most of the streamflow measurements.

A variety of methods are available in FEQUTL for computation of the kinetic energy coefficient (α) and momentum coefficient (β) within the subsections of a cross section. The simplest method is to assume that α and β are both equal to 1.0 in each subsection. Another alternative is to apply a computation known as USGSBETA in FEQUTL input. The coefficients are then computed with the formulas

$$\alpha = 14.8n + 0.884$$

and

$$\beta = 1 + 0.3467\,(\alpha - 1)\,,$$

where n is Manning's n for the subsection. The third method is to apply a computation known as NEWBETA in FEQUTL input. The depth-averaged velocities obtained by locally applying Manning's equation at each point in the cross section are integrated in the computation. Sensitivity tests were done by applying all three methods of computing the coefficients, and no differences were apparent in the simulated stage and discharge results. The NEWBETA method was applied to the calibration and verification simulations.

The relative importance of cross-sectional geometry in producing reliable and accurate results was tested by replacing 8 of the 17 measured cross sections with interpolated sections. Very little difference was observed between the results of the base simulation and the sensitivity simulation. However, in the reach just upstream from the Cenacle culvert, which is the site of many measured cross sections, the sensitivity results indicated changes. At low flows, the simulated water-surface elevation was sensitive to the cross sections used. The results using the interpolated sections were as much as 0.2 ft lower than the base simulation results. To further analyze the effects, another sensitivity simulation was performed where the cross sections were still interpolated, but the thalweg elevations were specified from the measured cross sections. This effectively preserved the thalweg profile. The results of this simulation more closely matched the base simulation results. Thus, the sensitivity to the profile of the thalweg seemed to be an important part of the sensitivity to cross-sectional geometry.

Sensitivity to the roughness coefficient, Manning's n, was tested by increasing and decreasing the base simulation value by 30 percent. In these sensitivity tests, all values of Manning's n in the study reach were increased or decreased simultaneously. A 30-percent increase in the roughness resulted in a decrease in elevation of about 0.2 ft at the peak at the Forest Preserve (site 1) and near Morris Court. A 30-percent decrease in the roughness resulted in an increase in elevation of about 0.3 ft at the peak at the Forest Preserve (site 1) and near Morris Court. In the reach upstream from Cenacle culvert, the elevation was not sensitive to changes in roughness since the discharge through the culvert controlled the elevation. In the reach downstream from Cenacle culvert, the increase in roughness caused a decrease in peak elevation of about 0.3 ft, whereas the decrease in roughness caused an increase in peak elevation of about 0.4 ft. The discharge results were not sensitive to roughness, inasmuch as discharge is forced at the upstream boundary.

Several sensitivity tests were made on the use of the control structures modeled in FEQUTL. The first test was to run the model with the culverts omitted. This simulation was done by replacing the culvert control structure with a simple equality-of-elevation relation in the FEQ input. When Morris Court culvert was omitted from the simulation, results did not show any apparent differences from the base simulation results except at Morris Court headwater (site 2), where the peak elevation decreased by about 0.2 ft. When the Cenacle culvert was omitted from the simulation, the total flow was unaffected but the proportion of flow in the main channel (at the Intermediate staff gage, site 6) and the Overbank (site 9) differed from the base simulation results. The peak flow at the

Intermediate staff gage (site 6) increased from 116 to 175 ft^3/s, whereas the peak flow in the Overbank (site 9) channel decreased from 64 to 5 ft^3/s. The water-surface elevation at the Intermediate staff gage (site 6) was lower by 1.3 ft, whereas downstream from the culvert, the elevation at the Departure-overbank junction (site 8) was higher by 0.1 ft.

Sensitivity to the roughness of the culvert overbank channel modeled in CHANRAT was tested by varying the roughness of the overbank while roughness in the main channel was kept constant. Results of the tests are shown in figure 13. The calibrated value of Manning's n in the overbank channel was 0.045, which is in the high end of the range of values for the channel type, and sensitivity analysis was done with values of 0.035 and 0.025. The comparison of the calibrated roughness (0.045) with the reduced roughness (0.035), showed that the peak flow in the Overbank (site 9) increased from 64 to 68 ft^3/s, whereas the peak elevation at the Intermediate staff gage (site 6) decreased by 0.1 ft. The decrease in roughness to 0.025 caused an increase of 8 ft^3/s in the Overbank (site 9), whereas the peak elevation at the Intermediate staff gage (site 6) decreased by about 0.3 ft.

The length and slope of the overbank channel can vary with flow conditions and are not necessarily obvious during low flow. The starting and ending points of the overbank channel were estimated from the cross sections around Cenacle culvert. From these points, the length and slope were calculated. If the start and end of the overbank channel were improperly estimated, then the calculation of length and slope would be in error. Sensitivity to such errors was tested by varying the length of the channel while holding the start and end elevations constant. The slope was recalculated to correspond to the length. A length of 220 ft was used in the verification simulation with a slope of 0.0069. Sensitivity analysis was done with a length of 330 ft, resulting in a slope of 0.0046; and a length of 110 ft, resulting in a slope of 0.014. The simulation results (fig. 14), were found to be only slightly sensitive to the relatively large changes in the length and slope of the overbank. The shorter, steeper channel caused an increase of 5 ft^3/s in the peak flow through the Overbank (site 9). The longer, less steep channel caused a decrease of 3 ft^3/s in the peak flow through the Overbank (site 9).

The assumption that the overbank channel is prismatic is a significant simplification. Actually, at some points, the channel is very wide and shallow; at other points, the channel is narrow and deep. Only one cross section can be used to represent a prismatic channel, although several were available from the field surveys. The sensitivity to the choice of cross section was tested, and results are shown in figure 15. The cross section used in the verification, the overbank portion of cross section 9, has the steepest banks of the available sections. The overbank portion of cross-section 6 is somewhat wider and shallower, whereas cross section 8 has a very wide and shallow overbank portion (fig. 8, sites 6, 8, and 9). The roughness, length, and slope were held constant in the sensitivity analysis. The results show that the wider, shallower channels simulated a higher peak flow through the Overbank (site 9). The shape of the channel also affects the shape of the hydrograph as well as the peak flow (fig. 15).

The area of the culvert is computed in FEQUTL from a cross section in the input. Sensitivity to the area of the culvert cross section was tested by creating culvert function tables where the cross section was increased and decreased by 25 percent in area, while the shape of the cross section was kept similar. Decreasing the area of the culvert cross section restricted flow through the main channel, diverting water to the Overbank (site 9). The peak flow at Intermediate staff gage (site 6) decreased by 25 ft^3/s, and the elevation rose by 0.2 ft. Increasing the area caused an 18-ft^3/s increase in peak flow at Intermediate staff gage (site 6), and the elevation dropped by 0.2 ft. For both increased and decreased areas, the effect on water-surface elevation was throughout and following the peak flow (fig. 16), but was not significant before the peak flow.

EVALUATION OF THE MODEL

For the study river reach, the model simulations were accurate and reliable. The model accurately simulated stage and discharge through the culvert and in the overbank, indicating that the channel geometry, roughness coefficients, and boundary and initial conditions were accurately represented.

The verification demonstrates the difficulty of simulating a stream reach small enough to be affected by microfeatures, such as debris jams and small bottom elevation variations that create riffles at low flow. The intent of this study was not to demonstrate the detailed representation of features that would not or could not be simulated in most unsteady-flow

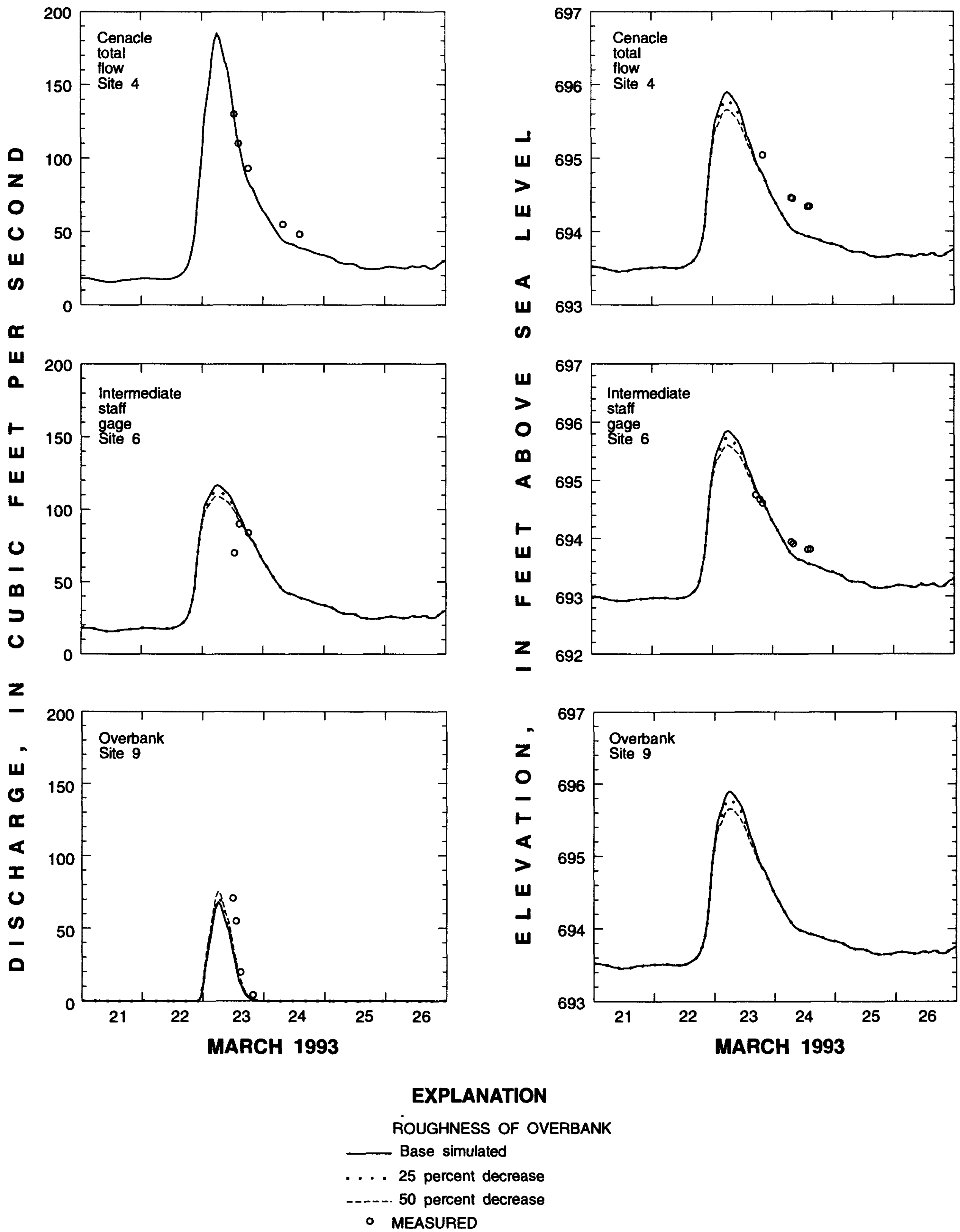


Figure 13. Effect of decreasing the overbank roughness coefficient by 25 and 50 percent on stage and discharge at sites surrounding Cenacle culvert on Spring Brook, a tributary to the West Branch Du Page River, in Illinois. (Site numbers are referenced to table 1.)

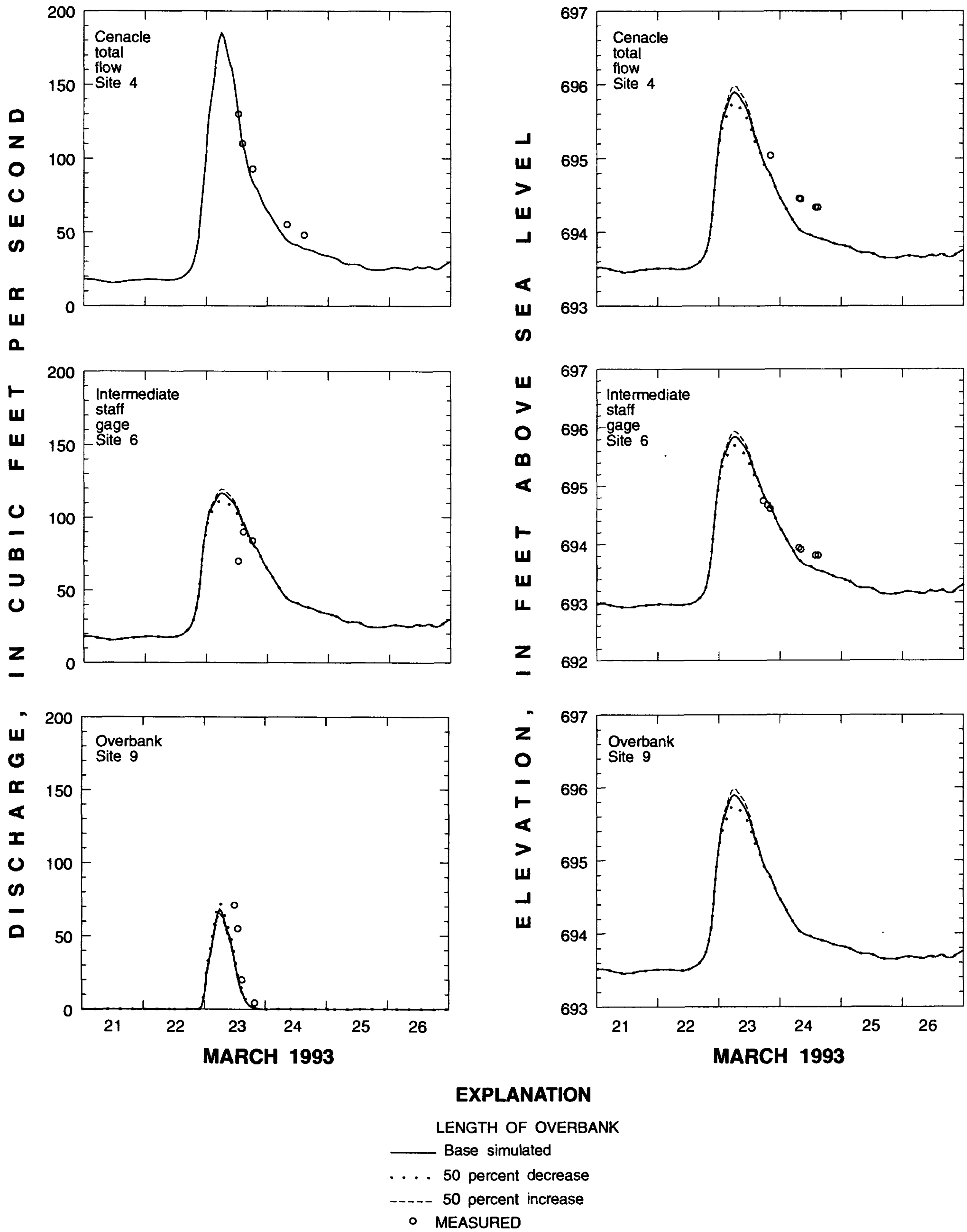


Figure 14. Effect of varying the overbank length on stage and discharge at sites surrounding Cenacle culvert on Spring Brook, a tributary to the West Branch Du Page River, in Illinois. (Site numbers are referenced to table 1.)

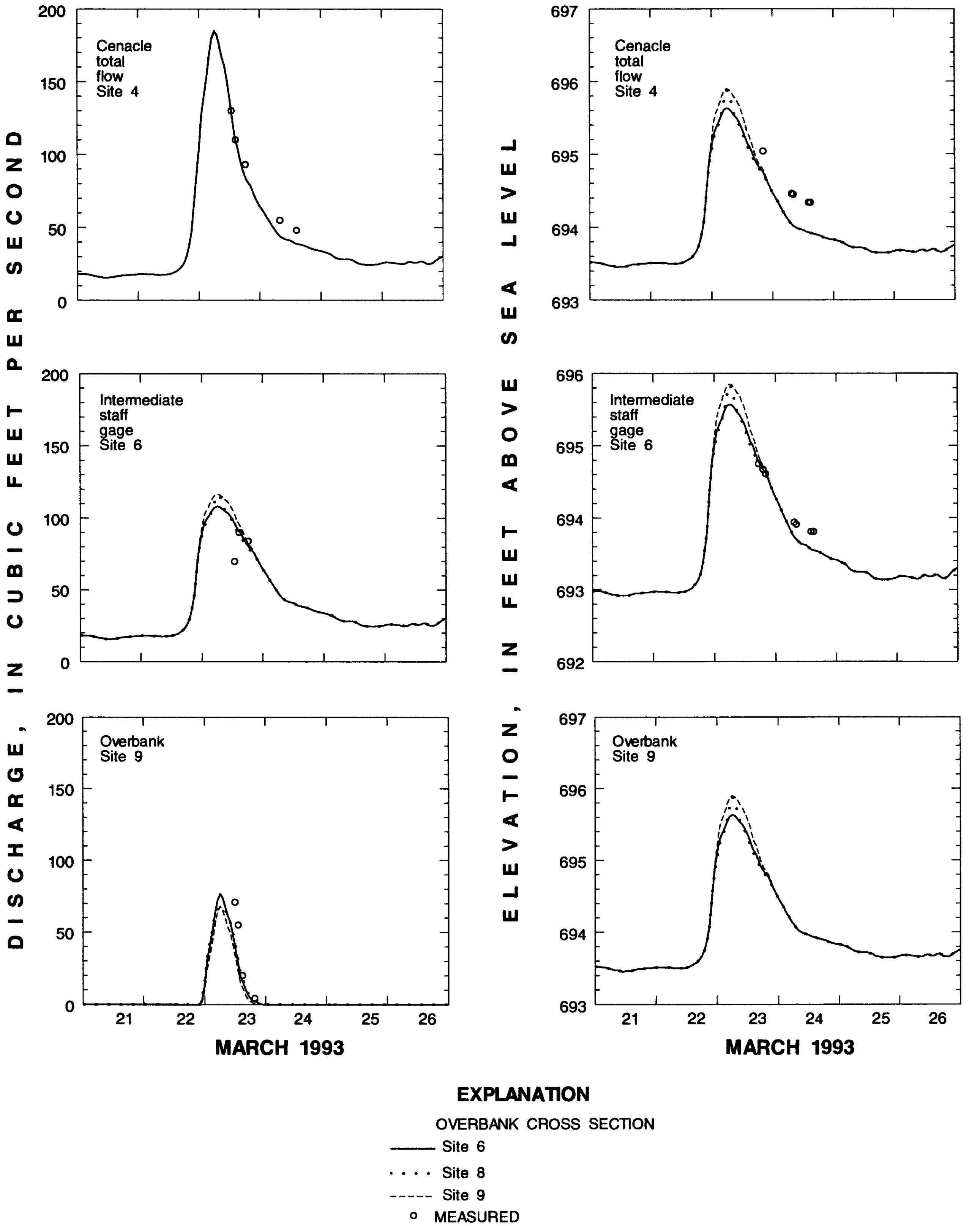
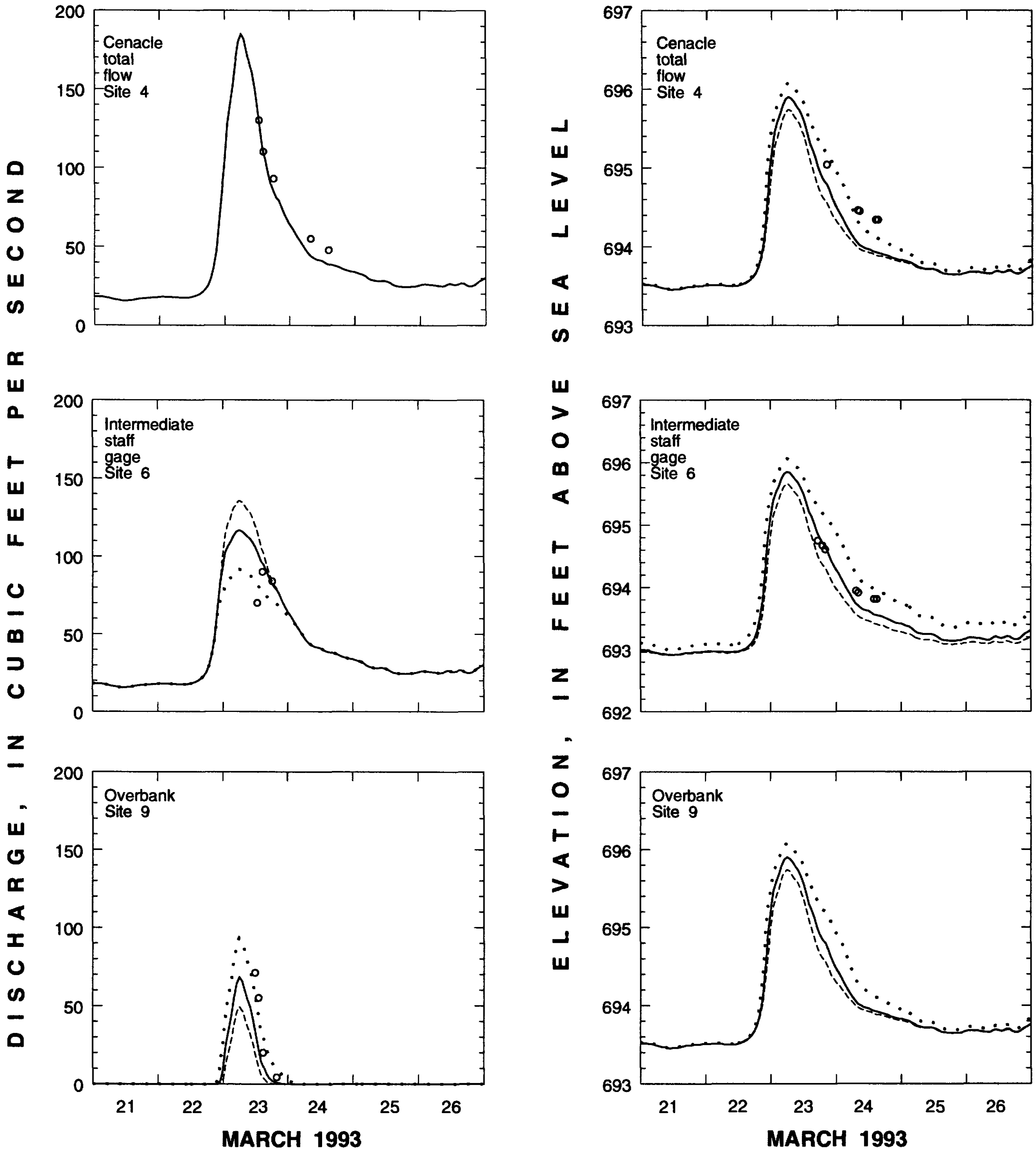


Figure 15. Effect of overbank cross-section selection on stage and discharge at sites surrounding Cenacle culvert on Spring Brook, a tributary to the West Branch Du Page River, in Illinois. (Site numbers are referenced to table 1.)



EXPLANATION

AREA OF CULVERT OPENING

——— Base simulated

. . . . 25 percent decrease

- - - - - 25 percent increase

o MEASURED

Figure 16. Effect of culvert size on stage and discharge at sites surrounding Cenacle culvert on Spring Brook, a tributary to the West Branch Du Page River, in Illinois. (Site numbers are referenced to table 1.)

studies, but to verify that the results of using the typical assumptions and scale of culvert and overbank representations are appropriate and sufficient for representation of the stream and its features in FEQ.

For this study, total flows and water-surface elevations were routed accurately and were properly simulated. The division of flow between the culvert and the overbank was the most difficult to simulate. For the calibration period, the flow was divided correctly, and the roughness coefficient selected at that time was maintained for the other periods. For the verification periods, measured flow exceeded the simulated flow in the overbank for all but two measurements because of the debris jams in the culvert that were not represented in the model. Further evidence of the significance of the overbank section is provided from the sensitivity analysis of the overbank results to the roughness coefficient. A large range of Manning's n may be assumed for the grassy swale of the overbank depending on the length, flexibility, and smoothness of the grass. Using interpolated cross sections may be acceptable if the bed profile of the stream is preserved.

SUMMARY AND CONCLUSIONS

A one-dimensional, unsteady-flow model, FEQ model, based on the de Saint Venant equations for dynamic flow in open channels was verified on a small stream in northeastern Illinois. The reach of the stream used for the study, Spring Brook, a tributary to the West Branch Du Page River, is 0.75 mi long with two culverts, one often with overbank flow. Streamflow data were collected at 10 sites along the reach during three high-flow periods: December 30, 1992–January 6, 1993, March 23–24, 1993, and June 7–9, 1993. Data collected during the June period was used to calibrate the model; data from the December–January and March periods were used for verification. The periods were simulated with FEQ, and the simulation results were compared graphically with the measured streamflow data. Errors in simulated stage and discharge were relatively small except when debris clogged the culvert.

A sensitivity analysis of the physical and computational model parameters also was done. The model was insensitive to replacement of measured cross sections with interpolated cross sections, especially if the measured thalweg elevation was preserved. Changes in the slope and length of the overbank section, as well as the chosen representative measured cross section, caused only slight changes in the peak stage and discharge of the simulation results. Misrepresentation of the culvert area caused large discrepancies in the simulated high flows in the vicinity of that culvert, whereas the simulated low flows were unaffected. The simulated elevations were more equally affected throughout all flows and especially on the falling limb of the stage hydrograph, by as much as 0.4 ft. The FEQ model and the FEQUTL model routines for simulating culvert and overbank flow were evaluated as accurate and effective for this application.

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