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HYDRAULICS
LABORATORY

TECHNICAL REPORT HL-92-8

EFFECTS OF A PROPOSED BUSHY PARK ENTRANCE CANAL RELOCATION, COOPER RIVER, SOUTH CAROLINA

by

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August 1992

Final Report

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Prepared for US Army Engineer District, Charleston
Charleston, South Carolina 29402-0919

26724640

TA7 W34 no. HL-92-8 c.3

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

Public reporting burden for this collection of information is estimated to average 1 hour per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestions for reducing this burden, to Washington Headquarters Services, Directorate for Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302, and to the Office of Management and Budget, Paperwork Reduction Project (0704-0188), Washington, DC 20503.

1. AGENCY USE ONLY (Leave blank)	2. REPORT DATE	3. REPORT TYPE AND DATES COVERED
	August 1992	Final report

4. TITLE AND SUBTITLE

Effects of a Proposed Bushy Park Entrance Canal Relocation, Cooper River, South Carolina

5. FUNDING NUMBERS

6. AUTHOR(S)

Allen M. Teeter

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)

USAE Waterways Experiment Station, Hydraulics Laboratory, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER

Technical Report HL-92-8

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)

USAE District, Charleston, PO Box 919, Charleston, SC 29402-0919

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES

Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT

Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

The effect of a proposed entrance canal on tidal flushing of Bushy Park Reservoir, 27 km north of Charleston, SC, was tested using a numerical model, and a canal cross section that would maintain present flushing was developed by trial. A previously developed two-dimensional model computed dynamic flow velocities, vertical velocities, and water-surface elevations for two verification periods, and for various entrance canal cross sections. The model was verified to tidal volume exchanges and water levels measured in the field. No data were available to verify vertical velocities. The model was then used to test a number of entrance canal geometries (width and depth) for the proposed relocation, and the reservoir tidal volume exchanges and flushing were compared to existing conditions. Existing tidal flushing can be maintained in Bushy Park Reservoir by constructing an upstream entrance canal of sufficient size. Recent ocean chloride intrusion data were used to confirm that the proposed site for the Bushy Park entrance canal would effectively reduce chloride

(Continued)

14. SUBJECT TERMS			15. NUMBER OF PAGES
Back River Reservoir	Cooper River	Tidal flushing	58
Bushy Park Reservoir	Reservoir flushing		16. PRICE CODE
Charleston Harbor	Salinity intrusion		

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED		

13. ABSTRACT (Continued).

intrusions. Analysis of previous intrusion events indicated that relocation of the Bushy Park entrance canal to the proposed site near Mempkin will eliminate the salinity intrusion events if other conditions (weekly inflow schedules for instance) remain the same.

PREFACE

The analyses of reservoir flushing and salinity intrusion events for the proposed Bushy Park Entrance Canal relocation were performed for the US Army Engineer District, Charleston (SAC). Studies were performed in the Hydraulics Laboratory (HL) of the US Army Engineer Waterways Experiment Station (WES) during the period August 1989 to December 1990 under the general supervision of Messrs. Frank A. Herrmann, Jr., Director, HL; Richard A. Sager, Assistant Director, HL; William H. McAnally, Jr., Chief, Estuaries Division, HL; and George M. Fisackerly, Chief, Estuarine Processes Branch, Estuaries Division.

The study was conducted and this report prepared by Mr. Allen M. Teeter, Estuarine Processes Branch. Mr. Walter Pankow, Estuarine Processes Branch, assisted in the preparation of the report. Mr. Howard A. Benson, Estuarine Processes Branch, was the on-site field engineer for the dye dispersion tests, and assisted in the design of the field tests. The dye tests were conducted by the US Geological Survey (USGS), Columbia, SC, office under contract with SAC. Mr. Thad Pratt, Estuarine Processes Branch, retrieved monitoring data from the USGS Columbia computer system for analysis, and operated the analytical salinity model used in this study. Dr. Kurt Getsinger, Environmental Laboratory, WES, conducted tests on dye uptake by water hyacinths. Mrs. Marsha C. Gay, Information Technology Laboratory, WES, edited this report.

The SAC contact person was Mr. James Joslin.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
miles (US statute)	1.6093	kilometres

EFFECTS OF A PROPOSED BUSHY PARK ENTRANCE CANAL RELOCATION, COOPER RIVER, SOUTH CAROLINA

PART I: INTRODUCTION

Background

1. In 1985, the US Army Engineer District, Charleston, reversed a 1942 flow diversion and rediverted most of the 442-cu m/sec Cooper River inflow back to the Santee River, leaving about 127 cu m/sec weekly average inflow from the Pinopolis Dam. The purpose of the rediversion was to reduce shoaling in Charleston Harbor, and initial indications are that expected shoaling reductions will be met or exceeded. Since rediversion, another concern of the Charleston District has become the protection of the Bushy Park Reservoir from occasional salinity intrusion into the entrance canal. Bushy Park is a tidal, freshwater reservoir located in the upper reaches of the Cooper River (Figure 1). About a dozen intrusion events have occurred, generally correlated to times of tidal high water and prolonged zero discharge from Pinopolis, high tidal amplitudes, and/or fluctuations in mean tide level. An alert system is in place to sense salinity intrusions, and to order supplemental freshwater inflow releases by the State of South Carolina, the operators of the hydroplant at the Pinopolis Dam. As a permanent structural solution, the Charleston District proposed a new facility to be added to the authorized Cooper River Rediversion Project consisting of a canal to relocate the entrance upstream to a point above the limits of salinity intrusion (US Army Engineer District, Charleston, 1983). A concern in the design of a relocated entrance canal is its possible effects on the flushing characteristics of the reservoir.

2. The US Army Engineer Waterways Experiment Station (WES) has assisted the Charleston District on a number of studies concerning the Charleston Harbor and Cooper River in support of the Cooper River rediversion. WES first became involved during the mid-1940's as consultant on the entrance to the Bushy Park reservoir at about river mile 43 on the Cooper River. During the 1950's through the 1970's, two physical hydraulic models were used at WES to conduct a series of studies on both Charleston Harbor and Bushy Park Reservoir conditions related to the Cooper River rediversion (Benson 1976; Benson 1977;

and Benson and Boland 1977). Charleston Harbor conditions were monitored by WES both before and immediately after the Cooper River rediversion in 1985, and again during 1987 for comparison analysis (Teeter 1989). The studies addressed mixing and sediment flushing in the harbor, as well as the inflow, tidal, and meteorological effects on salinity conditions within the Bushy Park Reservoir. A schematic numerical model was developed to examine salinity intrusion and applied to predict harbor-deepening shoaling effects (Teeter and Pankow 1989). The US Geological Survey (USGS), Columbia, SC, office has installed and maintains a system of satellite-telemetered water level and water quality monitors, providing real-time data for a WES-designed salinity alert system. Background salinity conditions were previously compiled for the reservoir, which the Corps is committed to protect from salinity intrusion.

Purpose

3. The purpose of this study was to evaluate the proposed Bushy Park entrance canal relocation and corresponding effects on tidal volume exchanges, flushing rates, and salinity intrusion. Dye tests were performed to gage flushing rates, tide heights, and volumes for the present canal configuration; to establish baseline conditions for the reservoir; and to provide information for evaluation of reservoir flushing processes. The purpose of the tidal model tests was to evaluate the Bushy Park Reservoir entrance canal relocation and corresponding tidal volume exchanges and to develop an entrance canal geometry that would maintain tidal flushing at its current level. The purpose of salinity analysis was to evaluate the effectiveness of the proposed entrance canal location in protecting Bushy Park Reservoir from ocean salinity intrusions.

Approach and Scope

4. The following overall approach was used in this study:

   a. Field measurements were performed with USGS to obtain new data (dye dispersion, tidal volumes, salinity, etc.) in the existing entrance canal for two tidal conditions. This report briefly describes prototype data collection activities and presents selected field data and an analysis of reservoir flushing using a simple model. The model considered tidal exchanges and industrial withdrawals.

b. Numerical modeling was performed using an established numerical model to determine the effects of existing and proposed entrance canal geometry (length and cross section) on reservoir flushing and tidal volumes. This report summarizes numerical model verification and tests with alternate entrance canal dimensions and supplemental withdrawals. Observed and computed reservoir tidal volume exchanges and water levels are presented.

c. Specific conductance data were compiled from historical records to test rediversion salinity effects at the DuPont intake station located on the reservoir entrance canal and to extrapolate data from other stations upstream to determine extreme transient conditions at the proposed entrance canal location. Historical data were compiled for times of high chloride* readings in Bushy Park between 1 January 1986 and 1 August 1989. Select intrusion events were analyzed using an analytic model to predict the maximum chloride intrusion at the proposed entrance canal location.

The organization of the next three parts of this report follows this outline.

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* Salinity is a measure of the total dissolved salts in water, with ocean water having a salinity of about 34 ppt. Chloride content is a more consistent measure in low-salinity waters and is the measure of preference for Bushy Park water quality (paragraph 58). Depending on the context, both measures are used here.

PART II: PROTOTYPE SYSTEM FLUSHING

Site Description

5. Bushy Park Reservoir is located approximately 27 km north of Charleston, SC. Figure 1 shows the Cooper River and the Bushy Park Reservoir. The reservoir was completed in 1956 by closing the Back River with an earthen dam and dredging an intake canal to bring fresh water from the Cooper River. Bushy Park Reservoir is used primarily for industrial and municipal water supply.

6. The reservoir is about 14.5 km long from the existing entrance to the dam. The reservoir is surrounded by marsh except at the industrial site between the reservoir and the Cooper River. The entrance canal intersects the Cooper River at river mile (RM) 43 as measured from the harbor jetties. The proposed entrance canal would add about 3.2 km to the overall length of the reservoir, and would situate the reservoir entrance about 6.4 km farther upstream on the Cooper River. Figure 1 shows a schematic of the proposed canal.

7. The existing entrance canal has dimensions of about 55 m wide by 4.2 m deep to mean tide level (mtl), and has been stable since its construction. No dredging has ever been required, nor are there any indications of shoaling in the canal. Canal side slopes range from 1V on 2H to 1V on 4H.

8. Tides are semidiurnal with a mean tide range of 1.62 m at the Customs House in Charleston Harbor. The average tide range near the entrance to Bushy Park Reservoir on the Cooper River is about 0.89 m, and just inside the reservoir about 0.55 m.

9. The volume and surface area of the reservoir were estimated at 10 million cubic metres and 2.5 million square metres at mtl. For comparison, the volume and surface area of the existing entrance canal were estimated at 0.8 million cubic metres and 0.2 million square metres at mtl.

10. The withdrawals from Bushy Park Reservoir are substantial. Total withdrawal capacity is about 31 cu m/sec. The South Carolina Electric and Gas (SCE&G) withdrawal alone is about 24 cu m/sec, about 77 percent of the total. Characteristic times for 50 percent replacement for the reservoir were roughly estimated based on plug-flow withdrawal T_w and exponential tidal replacement times T_t as follows:

$$T_w = 0.5 \frac{V_o}{Q_w} = 3.6 \text{ tidal cycles} \tag{1}$$

$$T_t = -\ln (0.5) \frac{V_o}{V_p} = 5 \text{ tidal cycles} \tag{2}$$

where

V_o = volume of the reservoir to mtl (without the entrance channel = 10 million cubic metres)

Q_w = withdrawal rate

V_p = tidal prism (without the entrance channel = 1.4 million cubic metres)

The tidal prism was calculated by multiplying the surface area of the reservoir without the entrance channel (2.5 million square metres) by the average tidal range.

11. Based on these rough estimates of replacement times, it appears that the flushing action of withdrawals dominates in the main stem of the reservoir between the entrance and the withdrawal points. Dye tests reported in this Part and numerical simulations reported in Part III evaluated flushing in this area. In other back, off-channel areas of the reservoir, especially Foster Creek and the upper part of Back River, the largest flushing action comes from water level fluctuations, also evaluated in Part III.

Dye Test Procedures

12. Three dye tests were performed by USGS starting 29 June, 9 August, and 30 August 1989 and carried out over 3-day periods. The first test was used to establish procedures, but the quantity of dye injected was insufficient to trace with confidence. The second and third tests used 34 and 42 kg of Rhodamine-WT fluorescent dye, respectively. The third test produced the greatest quantity of usable dye concentration data.

13. Special monitoring sites were operated specifically for this study by USGS within Bushy Park Reservoir and are shown in Figure 1, Insert A. Not shown are sites 1-3, located on the Cooper River. Water level gage reading and gage datums from these sites were provided to WES by USGS. The datum

provided for site 8 appeared to be in error, and was adjusted by 0.3 m based on comparisons to nearby gages.

14. In addition to water level measurements, USGS measured discharges half-hourly at two locations (near sites 5 and 8) along the existing entrance canal over complete tidal cycles. Supplemental data included hourly and daily average inflows at Pinopolis provided by the Charleston District, and SCE&G provided daily average withdrawals.

15. Flows were measured by integrating current speeds over the two cross sections in the entrance canal. Cross-section depths were surveyed, and a tag line cable tied across at each range. Two boats were used at each range to measure currents half-hourly at 0.2 and 0.8 of the instantaneous depth and 3-m intervals along the tag line. Gurley current meters with audio output were used by USGS to measure current speeds. Raw flow data were processed by USGS, and results for flow versus time for the two ranges were provided to WES.

16. Flow measurements began near the end of ebb tide cycles. At the first indication that the flow had completely turned to flood, dye injection began. Dye was injected at site 4 over 5- to 10-min periods from a boat passing slowly back and forth across the canal. A long bright red cloud was thus immediately formed in the canal and rapidly expanded into the reservoir.

17. Three boats were used for hourly daylight dye sampling. After the first day of each test, two boats were used to sample dye. Samples were taken at surface, middepth, and bottom. Surface and bottom sampling points were 0.61 m below the surface and 0.61 m up from the bottom. Small water bottles were used to collect samples. At night, automatic samplers were used to collect samples at sites 6, 10, and 15.

18. Samples were analyzed for dye concentration in the field using a Turner Designs Model 10 fluorometer. Dye sample times, locations, and concentrations were provided to WES.

Results and Discussion

19. Selected data from the dye test starting 30 August 1989 are presented as follows. Figure 2 shows water level data for the entrance canal and the reservoir over the 3-day period of the third dye test. Stages in Figure 2 and succeeding figures are referred to the National Geodetic Vertical Datum

(NGVD). Site 6 had an hourly recording interval, while other sites had half-hourly intervals. Figure 3a shows the data from sites 8 and 15 plotted along with the average stage for the reservoir. Figure 3b, the stage difference between site 8 and site 15, shows that differences can be over 0.15 m during flood tide phases, and over 0.1 m during ebb tidal phases. These data were interpolated to 10-min intervals using a spline. Stage differences should be regarded as being relative since datums are unverified.

20. Raw dye data consist of concentrations at various points and times. Data were divided into two groups: sites 3-8 for the entrance canal and sites 9-15 for the reservoir proper. Figure 4 shows surface, middepth, and bottom concentrations measured 30 August-1 September 1989 for sites 9-15. Middepth samples were not always collected. Concentrations are reported in dye units based on fluorometer response to calibration dilutions.

21. No dye-decay or background corrections were applied to the data. Dye photodecomposition, chemical degradation, and loss by adsorption to sediments were deemed too small to be important for these dye tests. However, there might have been dye loss by aquatic plant absorption, and uptake tests were performed to gage this effect.

22. Tests on the uptake of Rhodamine-WT by water hyacinths, similar to those which surround the reservoir to depths of about 9 ft, were conducted at WES. Four small, well-rooted plants were crowded into three tanks, and three additional tanks were used for controls. Dye concentrations in the hyacinth tanks were reduced by 9 percent during the first day, and by 12 percent during the first 2 days compared to control tanks. Plant uptake of dye is thought to depend strongly on the growth state of the plants, which is difficult to assess both for the laboratory and the field. Hyacinths dominate in shallow areas of the Bushy Park Reservoir, while deep areas of greater volume and flow are free from the plants. Because of the difficulties applying the uptake test results to the field, no direct corrections were made to the dye data; but the uptake rates were used to establish a qualitative sense of results accuracy.

23. Canal flow data for the second and third tests are presented in Part III. Daily freshwater releases from the Pinopolis Dam near the time of the third dye test were as follows:

Date	Mean Discharge cu m/sec
8/26	73.6
8/27	110.9
8/28	251.7
8/29	202.5
8/30	88.9
8/31	69.6
9/1	93.1

The exact total withdrawal rate is not known for the period of the dye tests, but data obtained through the Bushy Park Water User's Association indicated that the largest withdrawal (24.1 cu m/sec daily average) was at capacity during the third test.

Analysis of Dye Data

Tidal prism model description

24. A simple model was used to analyze the dye data from the third dye test and to evaluate reservoir flushing. The model calculates the mass and concentration of a tracer over tidal cycles based on conservation principles. The model is similar to a previous tidal basin flushing model that has shown good agreement with physical hydraulic model results (Callaway 1981). Withdrawal losses were added to the model.

25. The assumption is made that the reservoir is well mixed. The tidal prism $V_p = V_h - V_\ell$ where V is volume, and the subscripts h and ℓ refer to tidal high and low water stages, respectively. Then $V_p = A_o R$ where A_o is the reservoir surface area and R is the tidal range; and (V_p - withdrawal) is the ebb flow volume through the entrance canal. If the reservoir withdrawal rate is Q_w and the tidal period is T, then the mass of tracer M in the reservoir at the end of ebb is:

$$M_\ell = M_h - C_h \left[A_o R + Q_w \frac{T}{2}\right] \tag{3}$$

where M_h and C_h are the mass and concentration, respectively, at the previous high water. The concentration of tracer does not change by dilution during the ebb tide (C_l = previous C_h). During the flood tide, the mass of tracer in the reservoir is not changed by the tidal flow, but is reduced by the withdrawal. Thus, at the end of the flood tide:

$$M_h = M_l - C_l Q_w \frac{T}{2} \tag{4}$$

and

$$C_h = M_h / V_h \tag{5}$$

The model is used to evaluate concentrations and masses of tracer over a series of tidal high and low waters in steps.

Dye data averaging

26. Data plots such as Figures 4a-c showed that the dye was not uniformly distributed in the reservoir, and that concentrations had considerable scatter in time as well. For comparison to model values, dye data were averaged over flood and ebb tidal phases. Sampling sites were located along the channel axis of the reservoir. Data were therefore averaged in space by first depth averaging and then weighing according to the site depth. Average concentrations were then normalized by the initial observed concentration C_o.

Results of analyses

27. Model analyses for reservoir flushing were performed both without withdrawals and with an assumed 28.3-cu m/sec withdrawal rate. Results of the two model analyses and for the averaged dye data are shown in Figure 5. Dye test results should be compared to model predictions that included withdrawals. The flushing shown by the dye tests was somewhat greater than the model predicted, but given the difficulties inherent in dye tests and the simplified assumptions of the model, the agreement was acceptable. The initial dip exhibited in the dye data was disregarded. The flushing times for 50 percent reduction in initial concentrations were interpolated as 1.8 and 2.4 tidal cycles for the dye data and the model that included withdrawals, respectively. The model flushing time for 50 percent reduction in initial concentrations with no withdrawals was not reached in the four tidal cycles for which data were available. Using an average tide to drive the model after

four tidal cycles, 50 percent reduction was reached in five tidal cycles. Thus the model indicated that the withdrawals reduced flushing times by about 50 percent. These calculated flushing times are consistent with the rough estimates of paragraph 10.

PART III: TIDAL MODEL FLUSHING TESTS

Description of Model Tests

28. The two-dimensional laterally averaged numerical model called FIne-grained Bed Sediments (FIBS) was used for this study. The model was previously applied to predict the effect of the 1.5-m Charleston Harbor channel deepening on shoaling (Teeter and Pankow 1989). However, model features that stress fine-grained bed sediments were not employed in this application.

29. The model numerically solved laterally averaged dynamic differential equations for flow continuity and horizontal momentum over the interval between the water surface and the channel bottom and along the length of the channel. The horizontal momentum equation included nonlinear advection and quadratic bottom friction terms. FIBS solves equations over finite elements using a method of weighted residuals. The flow domain was discretized as a series of elements with 15 computational points, configured as five roughly horizontal rows of three nodes. Each node consisted of a vertical column of five computational points. Breadth at each node varied vertically as a quadratic function of depth. Model equations were solved explicitly over elements and with time. Appendix A presents a brief description of the model.

30. The numerical mesh covered Charleston Harbor and the Cooper River from the jetties (river mile 0) to the Pinopolis Dam (river mile 57) at 1.6-km node or cross-section spacing. The mesh included 12.9 km of the Wando River and 25.7 km of the East Branch Cooper River, also at 1.6-km node spacing. Bushy Park Reservoir was represented in the model at 0.8-km node spacing from the entrance to the dam and also 4 km of Foster Creek.

31. The model was operated by specifying water-surface elevations at the ocean (river mile 0) and inflows at Pinopolis. Optionally, withdrawals were made in Bushy Park Reservoir at two points, near the SCE&G plant (the largest withdrawal) 8.8 km from the entrance and near the dam, as described later.

Model Operation

Prototype data

32. Water levels are monitored along the length of the Cooper River by

USGS. Figure 1 shows the location of stations used for model verification. In addition, special monitoring station sites were operated specifically for this study by USGS. Those sites are located within Bushy Park Reservoir, as shown in Figure 1, Insert A. In addition to water level measurements, USGS measured discharges half-hourly at two locations (sites 5 and 8) along the existing entrance canal over two separate tidal cycles. Because vertical velocity measurements are difficult to obtain, no vertical velocity data were available with which to verify the model.

33. Hourly and daily average inflows at Pinopolis Dam were provided by the Charleston District. SCE&G provided daily average withdrawals.

Verification procedure

34. Ocean boundary specifications were developed from hourly Customs House water level data, fit with a cubic spline. Water level values were extracted at 172-sec intervals (the model time-step), and multiplied by 0.9. The resulting boundary condition was found to produce reasonable water level results at the Customs House (river mile 9.5) in the model. Freshwater inflow specifications were developed from hourly Pinopolis Dam discharge readings smoothed by locally weighted regression.

35. The model was verified to two data periods. The first data period, 11-14 June 1988, corresponded to the onset of a salinity intrusion event examined in Part IV. This data period was used to verify the Cooper River water levels from the Customs House to Pimlico.

36. For the first data period, the model was first spun up from a quiescent, uniform water level to a dynamic condition for two tidal cycles. Results from those calculations were used to restart the model for a second set of calculations lasting five tidal cycles. The final three cycles of this run were used for comparison to field data.

37. The second data period, 29-31 August 1989, corresponded to an intensive data collection period by USGS as described in Part II. The data considered during this period came from Bushy Park Reservoir entrance to the dam. The second data period also included flow measurements within the entrance channel at sites 5 and 8. The procedure for operating the model was similar to that used for the first data period described earlier.

Plan tests

38. The proposed relocation route was used for all plan tests (Figure 1, Insert A). The first plan tested used the average cross-section

dimensions of the existing canal. Subsequent plan tests had cross-section dimensions covering a range to reproduce the existing tidal ranges and entrance canal flows. Plan tests used tidal and inflow conditions from the second data period (29-31 August 1989).

Canal Test Results and Discussion

Model verification

39. Figure 6 shows comparisons between model and prototype water levels for the first data period starting 12 June 1988. Note that data shown in Figure 6 for Mempkin were actually from Pimlico, 2 miles downstream. Mempkin tide gaging was only recently initiated and does not extend to this data period. Comparisons of Mempkin and Pimlico water levels for 1-7 May 1989 showed water level fluctuations to be almost identical, with the root mean squares (RMS) about the means differing by only 0.003 m. Figure 7 shows part of that comparison. It is difficult to judge phases using hourly data, but it appears that the two stations are nearly in phase. Because of the importance of Mempkin as the interception point for the proposed canal, model comparisons were made at this point using data from the Pimlico gage.

40. It was difficult to establish accurate descriptions for shallow, off-channel areas, especially at and above the Tee on both branches of the Cooper River. Many of these areas were formerly rice ponds and partially leveed from the river. These areas act as off-channel storage for flow but do not contribute directly to momentum of the flow. Satellite images and air photos were useful in defining top width near and below the Tee, but areas above Mempkin were not well known.

41. Pinopolis inflow is usually zero for about 8 hr a day, and otherwise varies widely. The model responded more rapidly than the prototype to rapid changes in Pinopolis inflow. This was the reason for smoothing hourly inflow data. The effect can be seen in Figure 6 at day 14.2 when low water was exaggerated in the model by falling inflows from Pinopolis. Previously mentioned uncertainties in off-channel storage were undoubtedly responsible for this effect.

42. Also note that DuPont Chemical water levels (Figure 6) were influenced by conditions within the reservoir, as will be discussed later. Overall, verification to the first data period was considered good.

43. Verification to the second data period starting 29 August 1989 encountered uncertainties in specifying reservoir withdrawals. First efforts used constant withdrawals based on withdrawal capacity, but were not successful at reproducing water levels. Further checks with the Bushy Park Water Users Association and with SCE&G indicated that withdrawals are variable during an average day. The daily average withdrawal from SCE&G during this period was 24 cu m/sec.

44. USGS flow data for the second data period complicated the withdrawal question by showing a mean or net flow of -0.79 cu m/sec, with the negative sign indicating the seaward direction (Figure 8). The flow data were not in agreement with the reported daily withdrawal, indicating that the withdrawal measured during the tidal cycle was very small or that it was offset by a decrease in Cooper River water level over that tidal cycle. During another intensive survey performed on 9 August 1989, the tidal flow record did exhibit the expected offset of about 31 cu m/sec corresponding to the withdrawal amount (Figure 9).

45. Hourly withdrawals were not known. To examine the effects of withdrawals on water levels and flows, three withdrawal scenarios were tested: no withdrawal, capacity withdrawals, and a variable withdrawal averaging the capacity. The prototype data generally fell between the constant no-withdrawal and capacity-withdrawal cases (Figures 10 and 11). Variable withdrawals were found to either increase or decrease water level fluctuations depending on timing. Several variable withdrawal schedules were tested. Figure 12 shows the scenario in which withdrawals were reduced to 8 cu m/sec between 2200 hr and 0600 hr, and the withdrawals during the rest of the day were such that the daily average withdrawal was 31 cu m/sec. This withdrawal scenario was used for all subsequent plan testing.

46. While the effects of withdrawals can be seen in the mean water levels, model tests of these withdrawal scenarios showed that the average tide range, as calculated from the root mean square deviations of water levels from the mean, was insensitive to withdrawal variations. For example, the average tide ranges for site 15 shown in Figures 10-12 are within 0.003 m for the last three full tidal cycles. Thus, even though water levels were not reproduced in the model exactly, the model verification to the second data period was considered to be good.

Plan tests

47. The first plan of the lengthened canal tested used the existing canal cross sections. The results are displayed in Figure 13. Model results critical to the tidal flushing of the entire reservoir and surrounding tidal marshes were the water level fluctuations at sites 8 and 15 (the two ends of the reservoir), and the flows in the entrance canal. Model results from the last three complete tidal cycles were used to compute water level and flow variances, and these were then converted to average tidal ranges and flow variabilities. Results of model base and plan tests are shown in the following tabulation:

Canal Condition		Three-Tidal-Cycle Average Values		
Width m	Depth m	Tide Range Site 8 m	Tide Range Site 15 m	Flow Variability RMS, Site 4 cu m/sec
		Existing Canal		
55	4.2	0.445	0.434	81.8
		Lengthened Canal		
55	4.2	0.366	0.379	63.6
65	5	0.380	0.391	66.8
75	5	0.394	0.408	67.5
95	5	0.404	0.418	70.9
65	6.5*	0.465	0.483	88.2
65	6**	0.442	0.461	83.0

* Includes deepened old canal + 2 m and channel at site 8 + 1 m.
** Includes deepened old canal + 1.5 m and channel at site 8 + 1.5 m.

The effect of canal lengthening (with existing cross section) was greater on flows (22 percent reduction) than on water level fluctuations (16 percent reduction). Even though the 95-m-wide by 5-m-deep planned canal had a much larger cross section (293 sq m) than the existing canal (164 sq m), tidal ranges and flows were not improved much. (It was assumed that it was feasible only to deepen the existing canal, since widening would involve bridge and levee modifications.)

48. Figure 14 shows model results for the most successful canal cross-section design: 65 m wide by 6 m deep. Figure 15 shows typical cross-sections for the proposed new canal, remaining portions of the existing entrance canal, and the revised reservoir.

Withdrawal Scheme Test

49. A withdrawal scheme to augment flushing with the smaller, existing entrance canal cross section was tested. Results of model tests using the existing canal dimensions for the proposed canal cross section indicated that the tidal component of flushing decreased by 22 percent compared to the existing condition (paragraph 47). Using half the existing flow variability (40.9 cu m/sec) as the average ebb flow over a tidal cycle, the tidal flushing component can be estimated to decrease by about 9 cu m/sec for the case of the lengthened canal using the existing cross section. Thus, a supplemental withdrawal of 9 cu m/sec would compensate for the difference in the cross-section area between the proposed and existing canal dimensions, neglecting any effects of the supplemental withdrawal on tides.

50. To test this result, the model was rerun for the following conditions: base (existing), lengthened canal with existing cross section, and lengthened canal with existing cross section including a 9-cu m/sec supplemental withdrawal at the Back River Dam. The model was spun up with 127-cu m/sec inflow at Pinopolis and 31-cu m/sec Bushy Park withdrawal for three tidal cycles. Model results, plotted as cumulative flushing volume (withdrawal and ebb tidal flow) in Figure 16 for two tidal cycles, confirm that a 9-cu m/sec supplemental withdrawal compensates for the reduced tidal flushing. The supplemental withdrawal allowed the lengthened canal flushing volume to catch up with the base condition at the end of the flood tidal phase. The model run was slightly less than two complete tidal cycles, so that this occurred only once (Figure 16).

51. Data from the model runs were also used to explore the feasibility of a gravity flow outlet by comparing the water-surface elevations across the Back River Dam. Figure 17 shows two tidal cycles of water levels on the river side and on the reservoir side, including the same three reservoir conditions described in paragraph 50. Tidal phase and amplitude were altered at the Bushy Park side of the dam by the lengthened canal, and were almost 180 degrees out of phase across the dam. The head difference across the dam was as much as a metre for the supplemental withdrawal test, as shown in Figure 18. The supplemental withdrawal was used to demonstrate (rather crudely) the effect of reservoir drawdown. Rough calculations indicate that an efficient weir of 20- to 30-m width (with flap gate) might be sufficient to

produce the desired 9-cu m/sec average supplemental withdrawal from the reservoir.

52. Preliminary design of the proposed entrance canal using recent Charleston District topographic information and cross sections at 61-m intervals indicated the oversized canal would require excavation of 2.08 million cubic metres as compared to 1.42 million cubic metres for the existing canal dimensions. Therefore, substituting a pumped or weir withdrawal for the oversized canal would save 0.65 million cubic yards of excavation.

53. The substitution of withdrawal for tidal volumes to achieve flushing would be most effective along the main stem of the reservoir. The relative effects of the tidal and withdrawal components may not be equivalent in backwater areas of Foster Creek and Back River, as noted earlier.

PART IV: SALINITY CONDITIONS

54. Data from intrusion events were used to drive an analytic model of salinity intrusion. Under reduced freshwater inflow conditions, ocean chlorides intrude farther upstream in the Cooper River. The reservoir has experienced occasional ocean chloride intrusion events since rediversion, and values have reached as high as 100 ppm. It is these intrusion events that have led the Charleston District to continue to consider structural measures to protect Bushy Park water quality and inflow scheduling options to improve flexibility of the hydroelectric plant operated by the State of South Carolina.

55. A Charleston District alert system, operated by USGS, senses chloride and water levels. Based on releases from the Pinopolis Dam over the previous several days, the system attempts to anticipate conditions conducive to intrusion events. The system automatically orders supplemental freshwater releases from the Pinopolis Dam when necessary to protect Bushy Park water quality. However, because of the time lag required to halt a chloride intrusion event, supplemental releases are only partially effective. Water level and specific conductance fluctuations along the length of the Cooper River are shown in Figure 19 for a typical intrusion event.

Canal Relocation Effectiveness

56. The effectiveness of the canal relocation was gaged by calculating salinities for previous intrusion events. Field data and an analytic model were used. The analytic model is a one-dimensional closed-form solution of an estuarine convection-diffusion equation which includes tidal convection effects (tide range and tidal velocity at the estuary mouth), freshwater inflow, and water depth (Headquarters, US Army Corps of Engineers, 1991). The model, which assumes uniform distributions of salinity and velocity and constant depth and width over cross sections, calculates longitudinal salinity profiles for given sets of conditions. The model was used to calculate salinities at the proposed entrance location for intrusion events. These values were corrected for the hydraulic effect of the proposed canal and converted to chlorides.

57. Existing data were used to project ocean chloride intrusion at the proposed entrance canal location. The critical stations are DuPont Chemical

and Mempkin for the existing and proposed canal sites, respectively. Seven extreme intrusion events were identified and used for analyses. Selected data were used as model input to calculate the longitudinal distributions of salinity, and other data were used for model/prototype comparison.

58. Conversions between specific conductance (field data), salinity (model data), and chloride concentration (water quality criteria) were required. Specific conductance values from the USGS gages were converted to salinity using standard oceanographic methods. These methods are inaccurate at low salinities, where ratios between ionic species vary, so another method was required. Chloride concentration is customarily used to assess water quality conditions in Bushy Park Reservoir. Specific conductance SC and chloride Cl values (134 data points) from previous studies in the Cooper River have been used to develop an empirical regression relationship:

$$Cl = 0.2398 \cdot SC - 4.2636 \tag{6}$$

where Cl is in ppm and SC is in microsiemens (micro-S) per cm. This relationship is applicable to river and estuarine waters with SC less than about 2,000 micro-S per cm, and has a standard error of ±32 ppm Cl associated with it. Chlorinities C in ppm can be calculated from salinities S in ppm using the oceanographic expression:

$$C = S/1.80665 \tag{7}$$

It was found that values of C and Cl were closely related, and that

$$Cl = 0.736 \cdot C \tag{8}$$

could be used, along with the other expressions described, to compare model and field results.

59. The model requires salinity values at a minimum of three locations as input. The ocean source salinity was specified as 34 ppt in the model at the mouth of the estuary. Salinity values from Mobay Chemical and Pimlico

stations were used for model input at the other two locations. Values were calculated for other stations, including the proposed relocation site for the entrance canal at Mempkin.

60. The model calculations of maximum salinity at the relocation site (Mempkin) were corrected to account for alteration in tidal excursions in the vicinity of the entrance canal. The flow in the Cooper River between the existing and proposed entrance canals would be altered by the canal relocation. During flood tide, flow that had previously entered Bushy Park would continue upstream and feed into the relocated canal. The correction was calculated as the product of the increased tidal excursion length and the longitudinal salinity gradient in the Cooper River. The excursion length increase was estimated as 2.78 km for the tide ranges and tidal prisms expected to occur during intrusion events.

Information Sources

61. USGS maintains for the Charleston District a number of real-time conductivity and water level stations (including other water quality parameters) along the Cooper River. Special short-term stations have also been installed and operated in certain areas of interest. Stations generally record data internally at 15-min intervals and upload data every 4 hr via satellite link to the USGS computer at Columbia, SC. Stations used in these study tasks, located in Figure 1, include Pimlico, DuPont Chemical, Goose Creek, Mobay Chemical, Customs House, and Army Depot. The DuPont Chemical station is located along the existing Bushy Park entrance canal. Figure 1 also shows Mempkin, the site of the proposed canal relocation. Conductivity and water level data were inspected and selectively downloaded from the USGS computer. Downloaded sensor data were averaged by hour and separated by category.

Results and Discussion

62. Table 1 lists daily maximum instantaneous conductance values at DuPont Chemical for days bracketing the seven worst intrusion events. The seven worst days, listed in the following tabulation, were used in the model analysis. Figure 20 shows three example longitudinal salinity distributions

Event Date	Data Source	Daily Maximum Chlorides, ppm Mobay* Chemical	Goose	DuPont Chemical	Pimlico*	Mempkin**
6/24/86	Field	1,930	229	56	40	NA
	Model	1,930	279	72	40	27
7/21/86	Field	3,076	242	101	55	NA
	Model	3,076	437	104	55	35
12/22/87	Field	1,000	101	74	31	NA
	Model	1,000	168	51	31	26
6/04/88	Field	2,909	217	90	37	NA
	Model	2,909	355	75	37	30
6/25/88	Field	5,553	315	69	36	NA
	Model	5,553	563	86	36	3
7/28/88	Field	1,823	185	118	37	NA
	Model	1,823	262	68	37	31
7/29/88	Field	1,585	537	76	37	NA
	Model	1,585	241	66	27	31

* Field data were used as model input.
** Corrected for tidal excursion length (see paragraph 60). NA = data not available.

calculated by the model. Model results and computed Cl values for field data are summarized in the tabulation. The average difference between field and model-calculated values for DuPont was 9 ppm, and the standard deviation of the differences was 24 ppm. Data from water years 1981-1983 previously established the background chloride concentration in the Cooper River inflow to vary monthly between 14 and 37 ppm, and average 26 ppm. The model calculations show that conditions at the proposed entrance canal would remain near normal (less than 35 ppm Cl) during similar future intrusion events, and confirm that the canal relocation would be effective in eliminating ocean chloride intrusions from the reservoir.

PART V: SUMMARY AND RECOMMENDATIONS

63. Dye tests were performed in Bushy Park Reservoir to evaluate reservoir flushing under the present reservoir configuration and conditions. Selected data are presented and analyzed herein. Flushing was found to be relatively rapid and analysis indicated that industrial withdrawals substantially enhanced this process. Observed flushing time for 50 percent replacement was 1.8 tidal cycles. Results were representative of conditions along the main axis of the reservoir where dye samples were collected and withdrawals are made.

64. A simple tidal flushing model with reservoir industrial withdrawals incorporated was in qualitative agreement with field dye test results. Withdrawals substantially improved the flushing action of the main axis of the reservoir between the entrance and the withdrawal points. The model indicated about a 50 percent improvement in flushing time due to the withdrawals. Dye tests carried out by the US Geological Survey (USGS) evaluated the rapid flushing of the reservoir main axis by the combined action of tidal exchanges and withdrawals. It follows that in other back areas of the reservoir, especially Foster Creek and Back River, the largest flushing action comes from water level fluctuations which were the focus of the numerical model study performed.

65. The effect of the proposed entrance canal on tidal flushing of Bushy Park Reservoir was tested using a numerical model, and a canal cross section that would maintain present flushing was developed by trial. A previously developed two-dimensional model (Teeter and Pankow 1989) computed dynamic flow velocities, vertical velocities, and water-surface elevations for two verification periods and for various entrance canal cross sections. The model was verified to tidal volume exchanges and water levels measured by USGS (vertical velocity calculations were not verified). Observed Charleston Customs House tides and Pinopolis discharges were used to drive the model boundaries. The model included the harbor channel configuration (which was being deepened), and reservoir withdrawals as they existed for the USGS field test. After verification, the model was used to test a number of entrance canal geometries (width and depth) for the proposed relocation, and the reservoir tidal volume exchanges and flushing were compared to existing conditions.

66. Existing tidal flushing can be maintained in Bushy Park Reservoir

by constructing an upstream entrance canal of sufficient size. The recommended canal cross section is 65 m wide by 6 m deep, with side slopes of about 1V on 3H, and a cross-sectional area of 272 sq m at mean tidal level. In addition, it is recommended that the remaining portion of the existing canal be deepened by +1.5 m to 5.65 m over a length of 3.2 km, and that the first 0.8 km of the reservoir be deepened by +1.5 m. A 9-cu m/sec withdrawal scheme was tested, and results indicated that such a pumped or gravity weir withdrawal might substitute for the effect of the canal enlargement, at least along the main stem of the reservoir.

67. Prior to finalizing the entrance canal relocation design, it is recommended that additional cross sections be taken at the two bridges crossing the existing canal, at the existing entrance, and at the proposed entrance. This supplemental information will be used to verify the assumptions made in establishing the model geometry. It is also recommended that, prior to the completion of the new canal design, an analysis should be made of possible sedimentation within the proposed canal. In addition, previous and/or new soil borings should be evaluated to confirm the stability of the 1V on 3H slope used in this study.

68. Recent ocean chloride intrusion data were used to confirm that the proposed site for the Bushy Park entrance canal would effectively reduce chloride intrusions. Analysis of previous intrusion events indicated that relocation of the Bushy Park entrance canal to the proposed site near Mempkin will eliminate the salinity intrusion events if other conditions (weekly inflow schedules, for instance) remain the same.

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Table 1

Salinity Intrusion Events

Year	Month	Days	Conductivity µS/cm
1986	June	18–26	223
			230
			255
			263
			298
			276
			248
			158
			176
1986	July	16–24	180
			185
			230
			308
			379
			426
			417
			257
			246
1987	Dec.	19–25	136
			178
			224
			312
			185
			156
			148
1988	May	30–31	257
	June	1–18	271
			177
			207
			258
			386
			296
			256
			191
			231
			244
			310
			311
			317
			232
			231
			246
			282
			259
			232
1988	July	25–31	226
	August	1–15	269
			482
			492
			340
			245
			277
			206
			224
			315
			264
			271
			240
			197
			241
			280
			276
			246
			262
			251
			218
			246

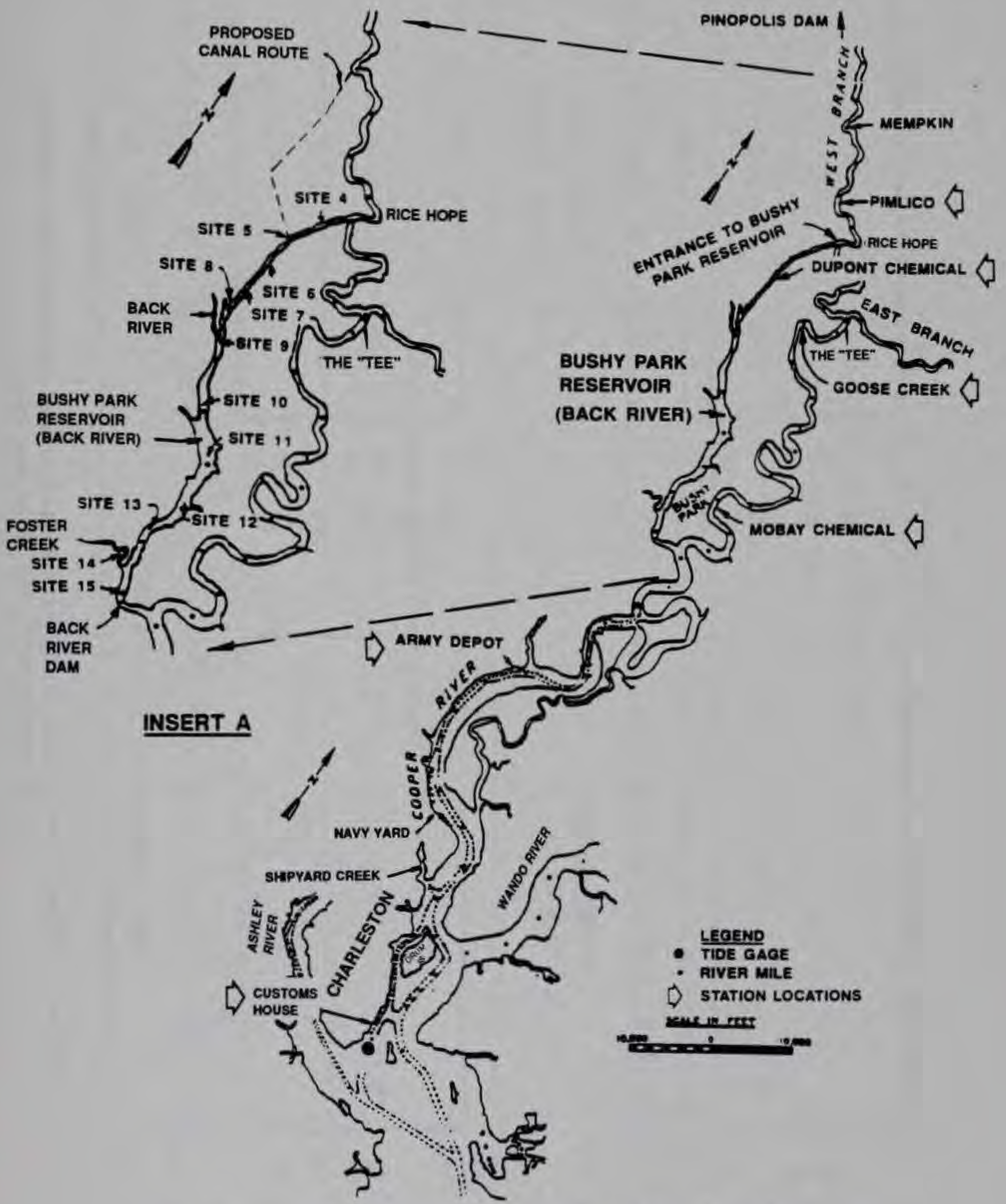


Figure 1. Vicinity map and sampling site locations

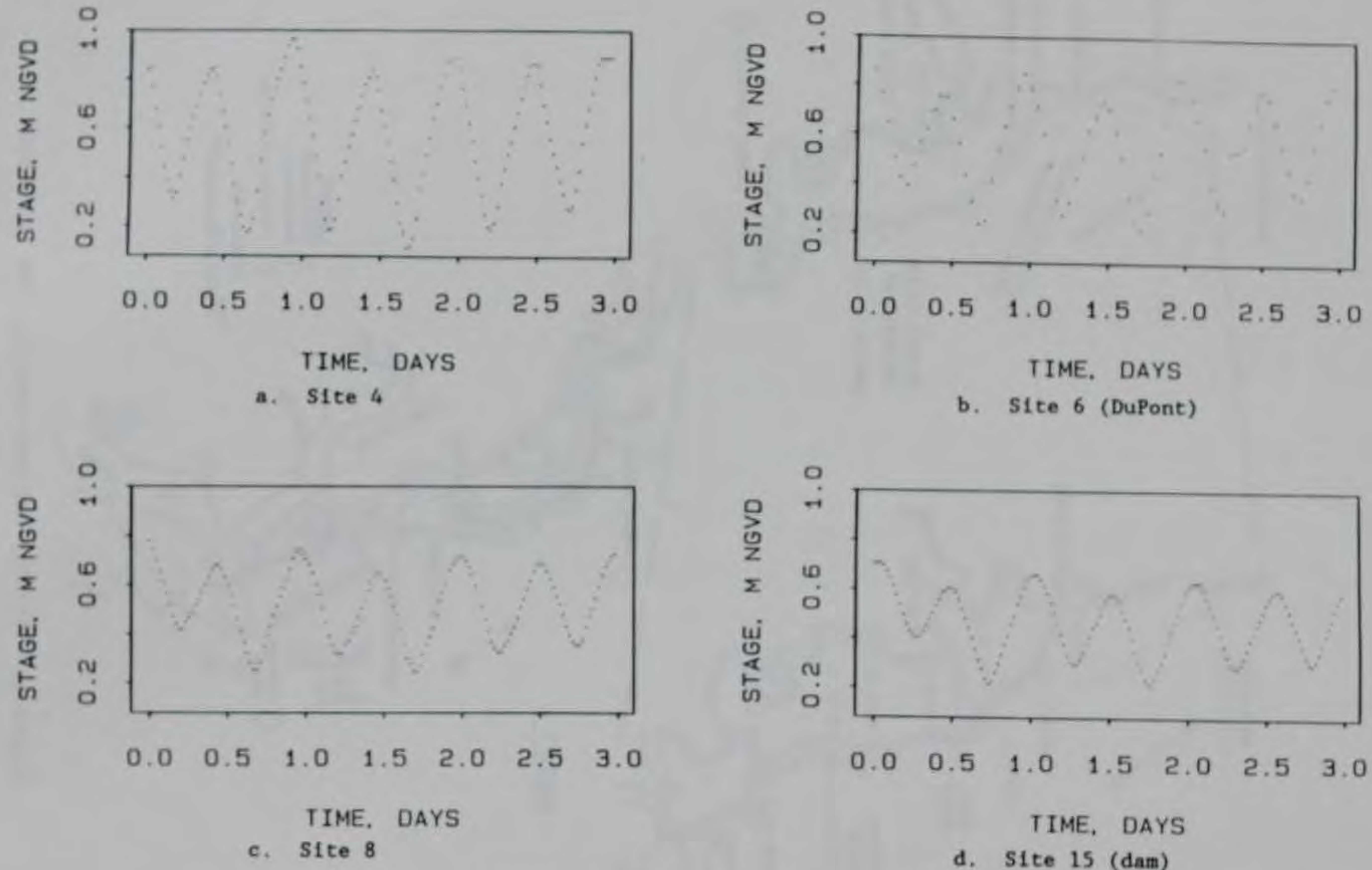
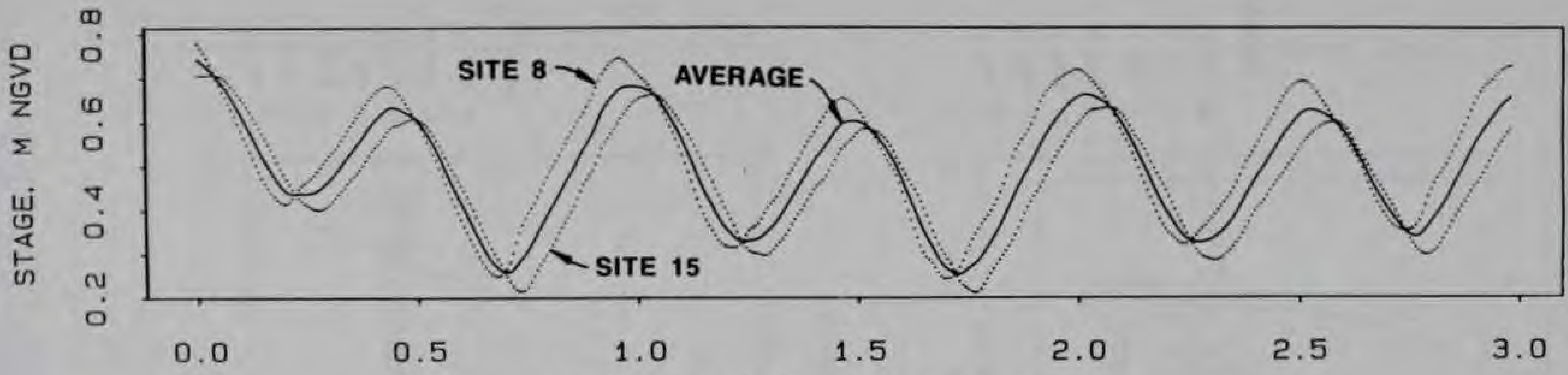
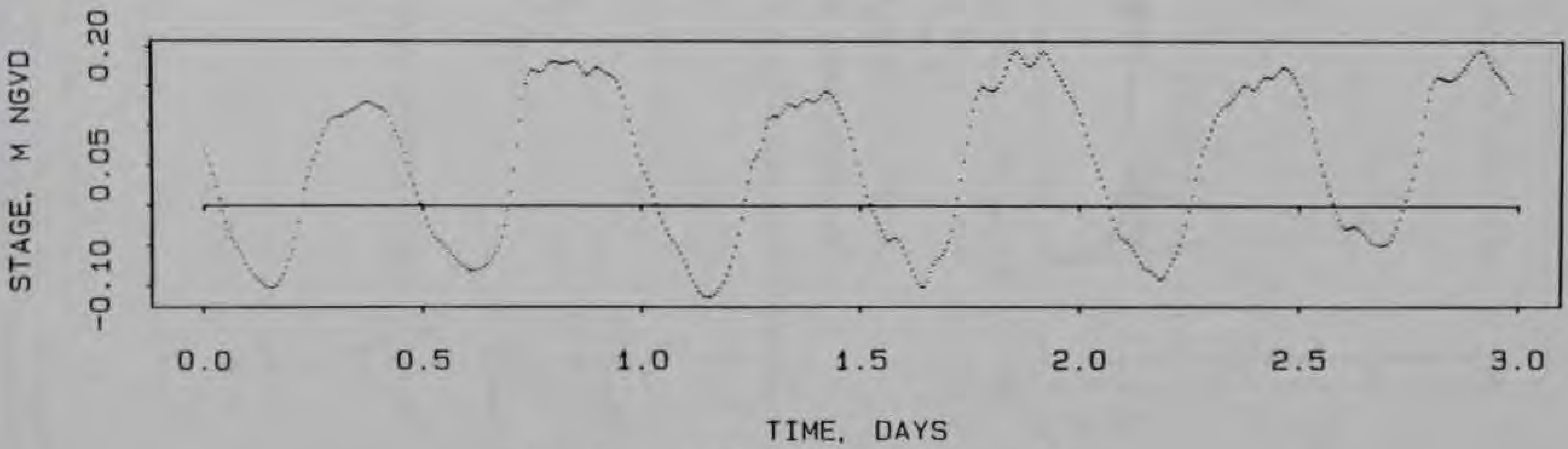


Figure 2. Water levels along Bushy Park Reservoir and entrance canal starting 30 August 1989 (the third dye test)

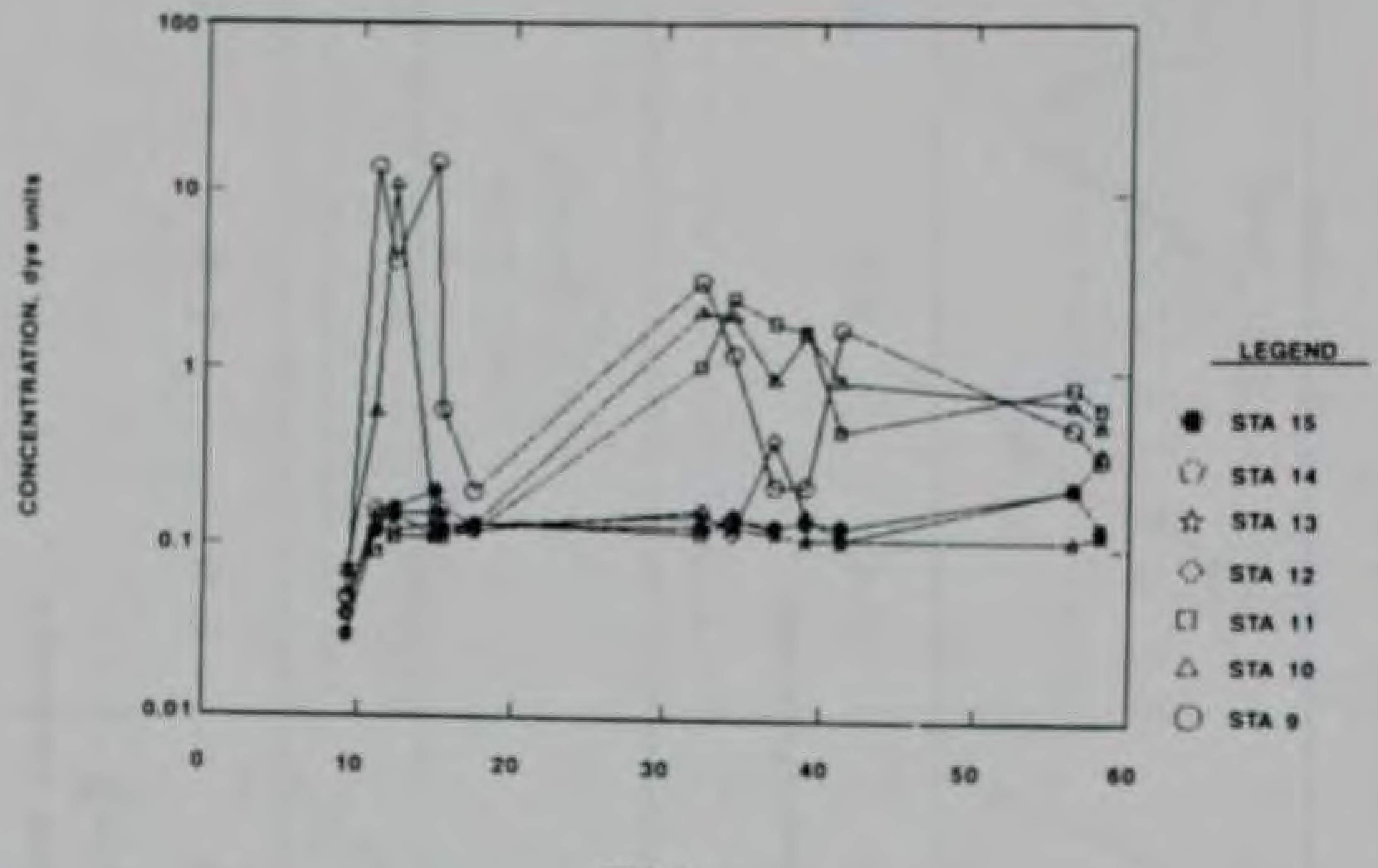


a. Sites 8 and 15 and average stage

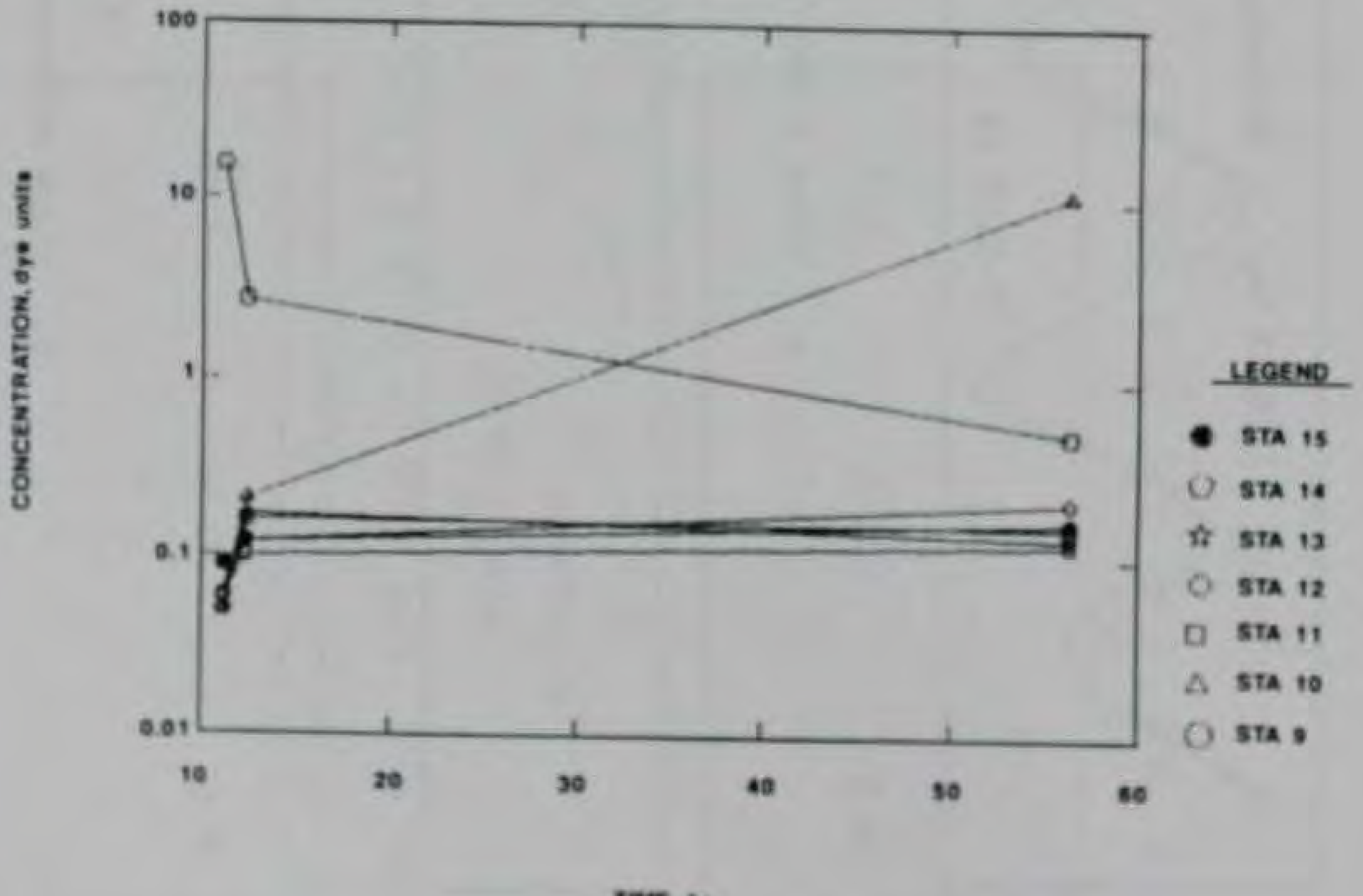


b. Stage difference along reservoir

Figure 3. Water levels and water level differences for Bushy Park Reservoir starting 30 August 1989

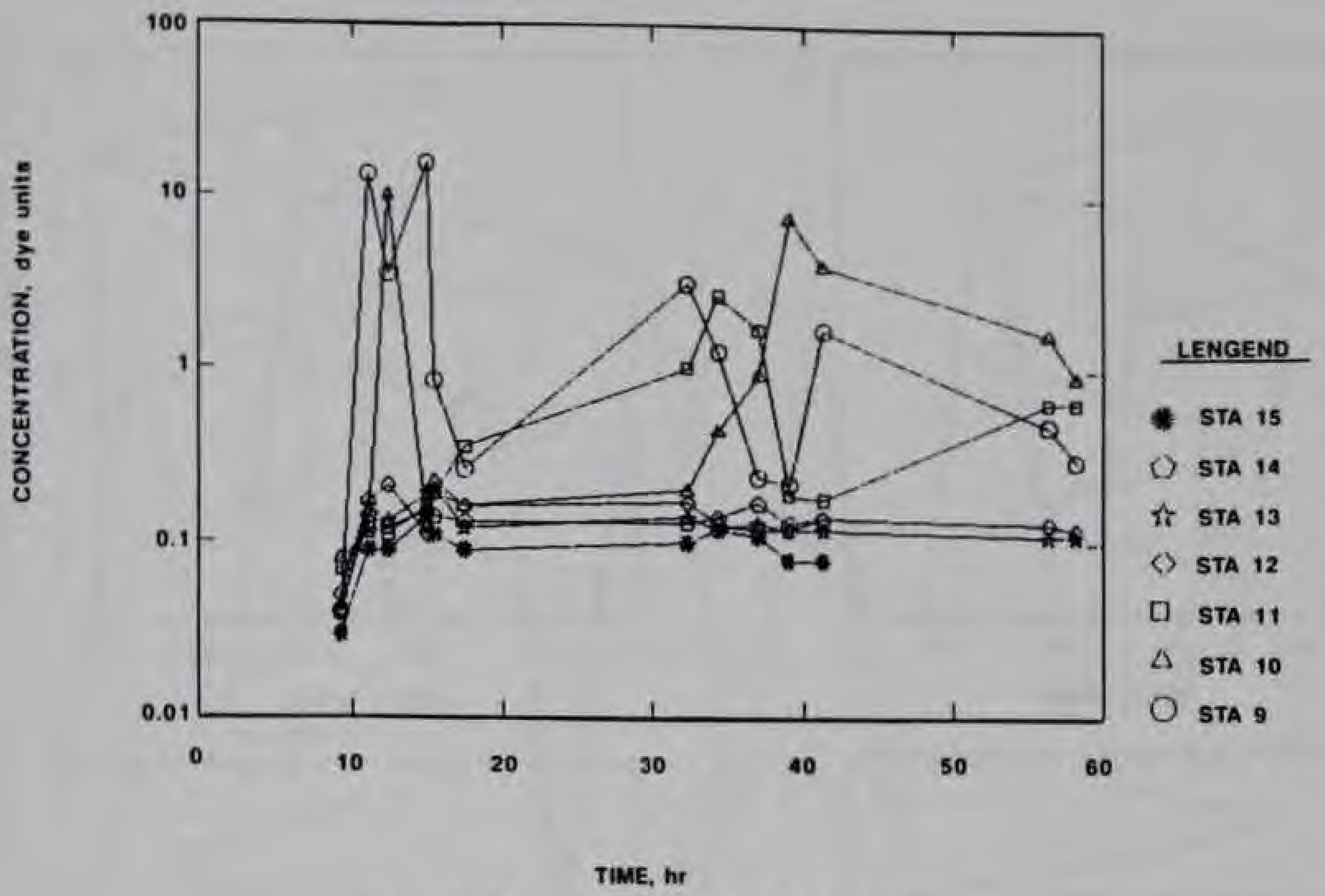


a. 2 ft under the surface



b. Middepth

Figure 4. Dye concentrations for sites 9-15 starting 30 August 1989 (times are approximate) (Continued)



c. 2 ft up from the bottom

Figure 4. (Concluded)

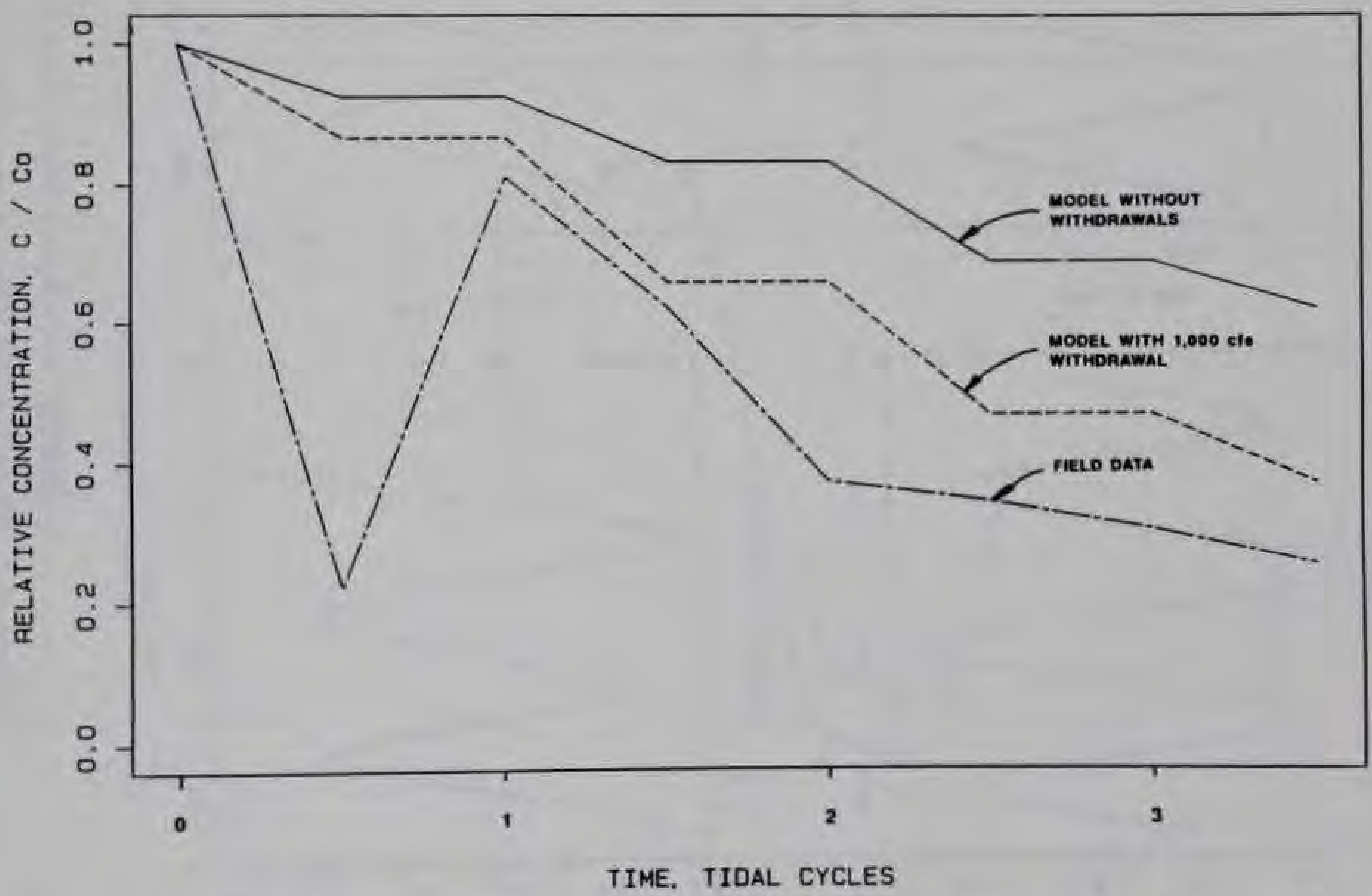


Figure 5. Model dye washout with and without withdrawals, and averaged field data for sites 9-15 starting at the 10:00-16:00-EST ebb tide of 30 August 1989

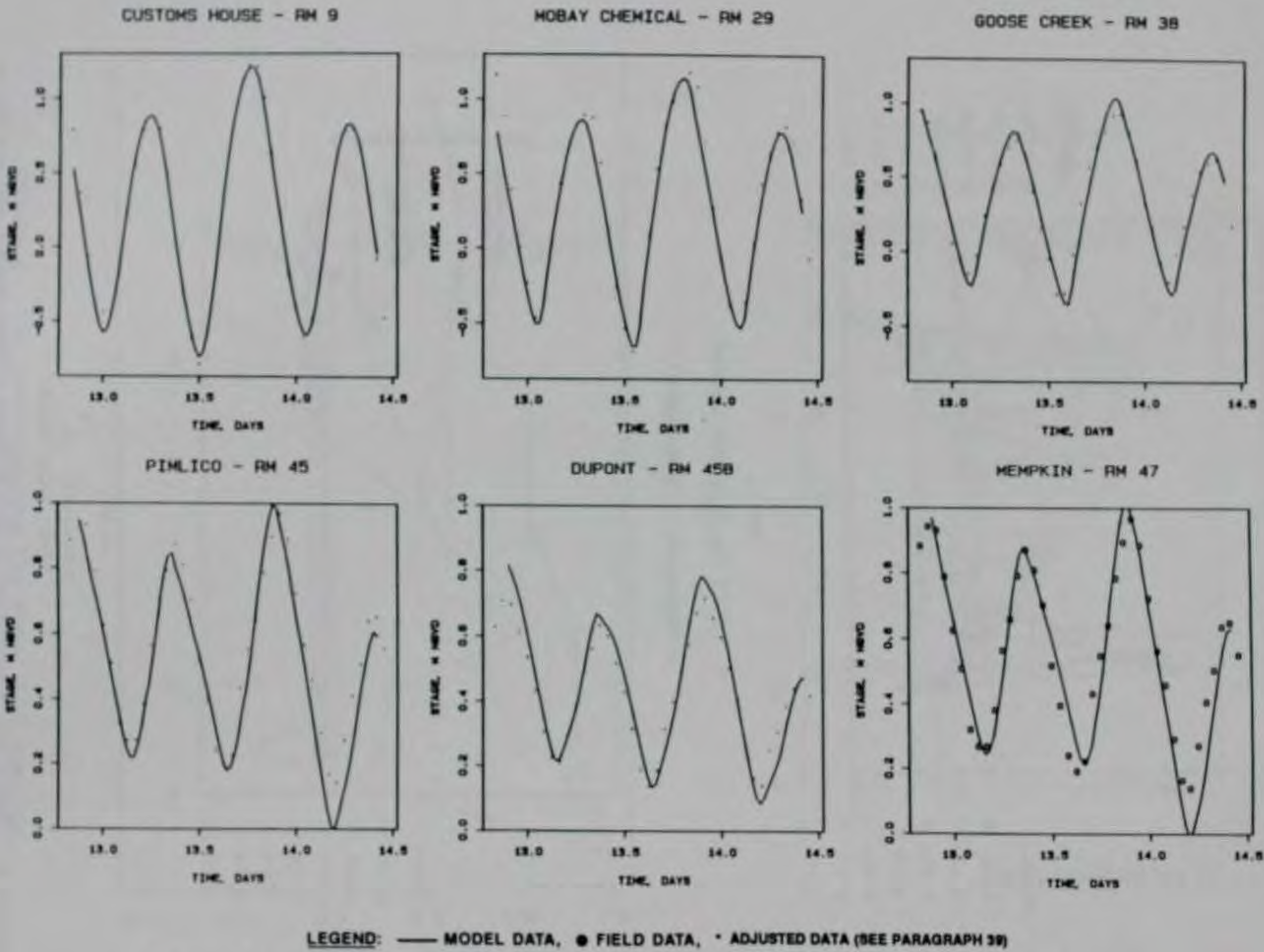


Figure 6. Results of the first data period verification starting 12 June 1988

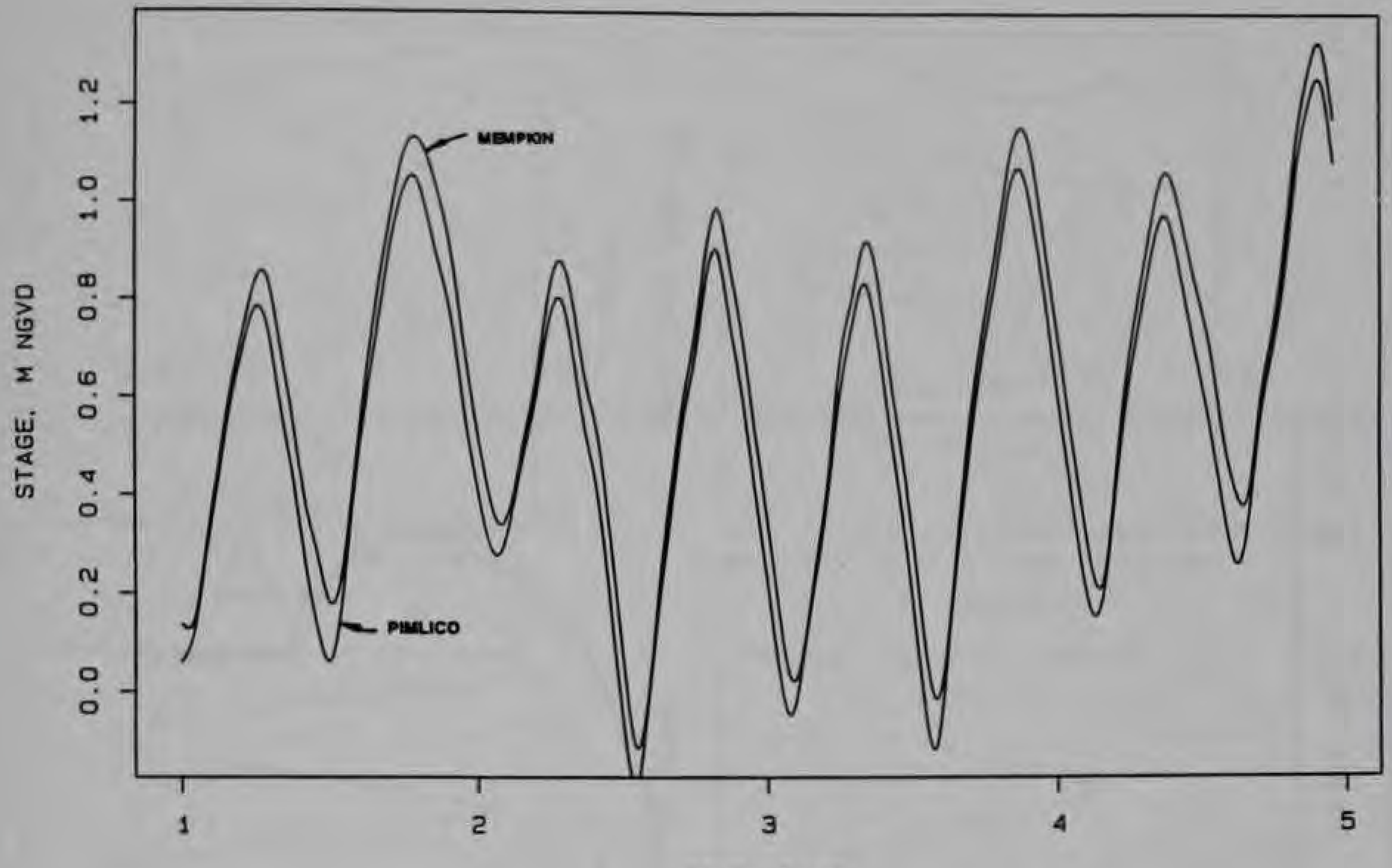


Figure 7. Comparison between Mempkin and Pimlico stages, May 1989

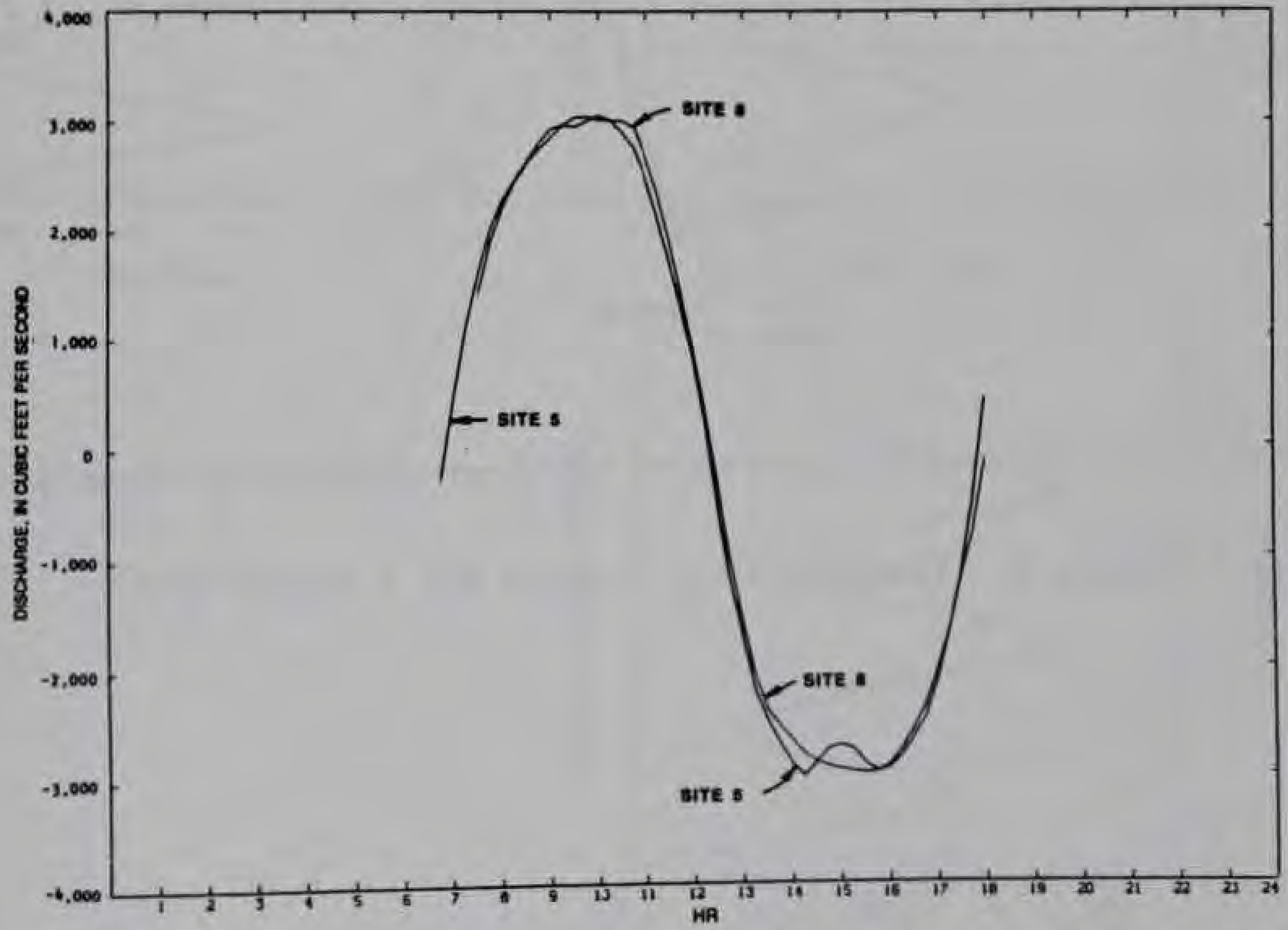


Figure 8. Flows at sites 5 and 8 for 30 August 1989

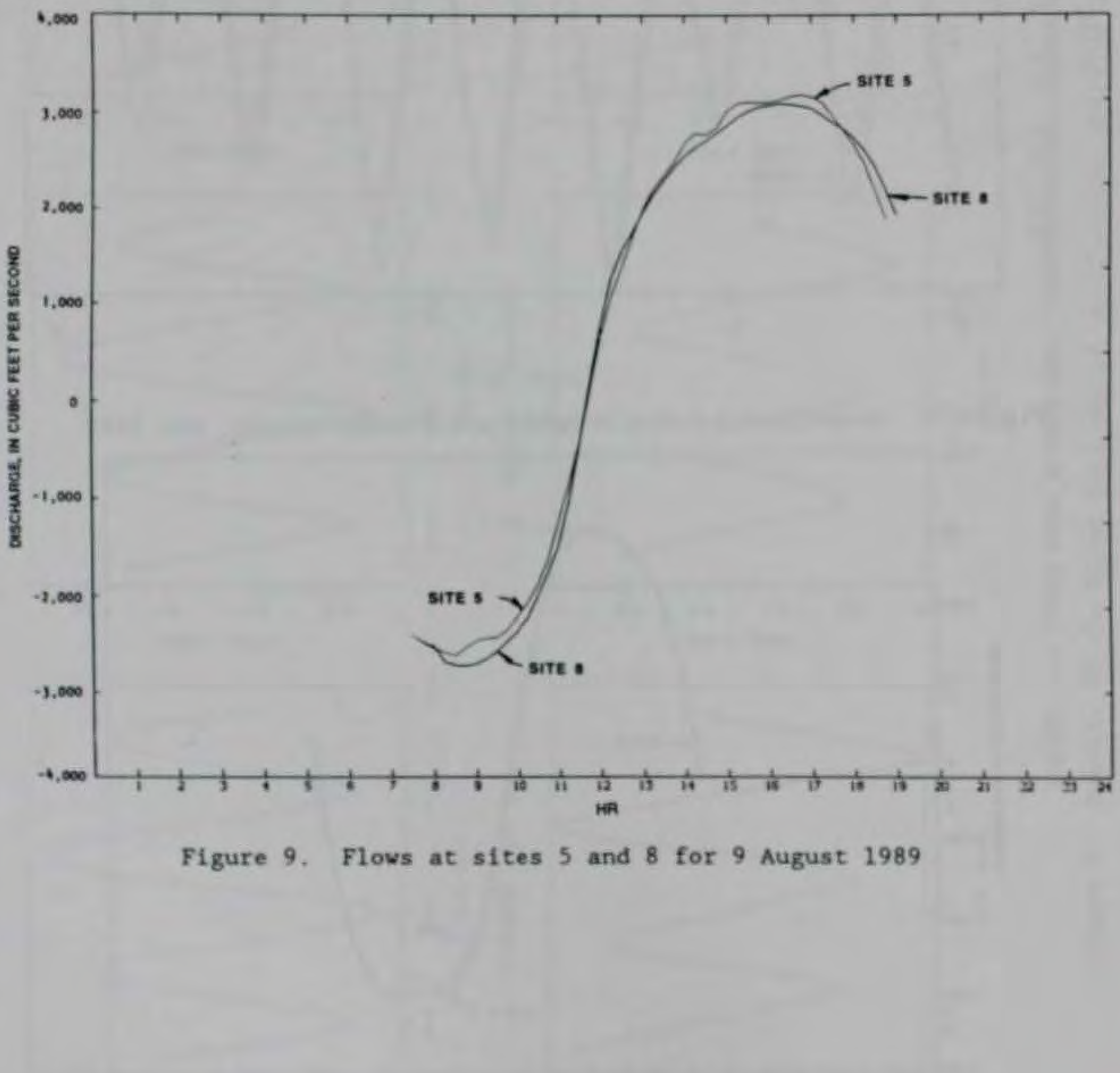


Figure 9. Flows at sites 5 and 8 for 9 August 1989

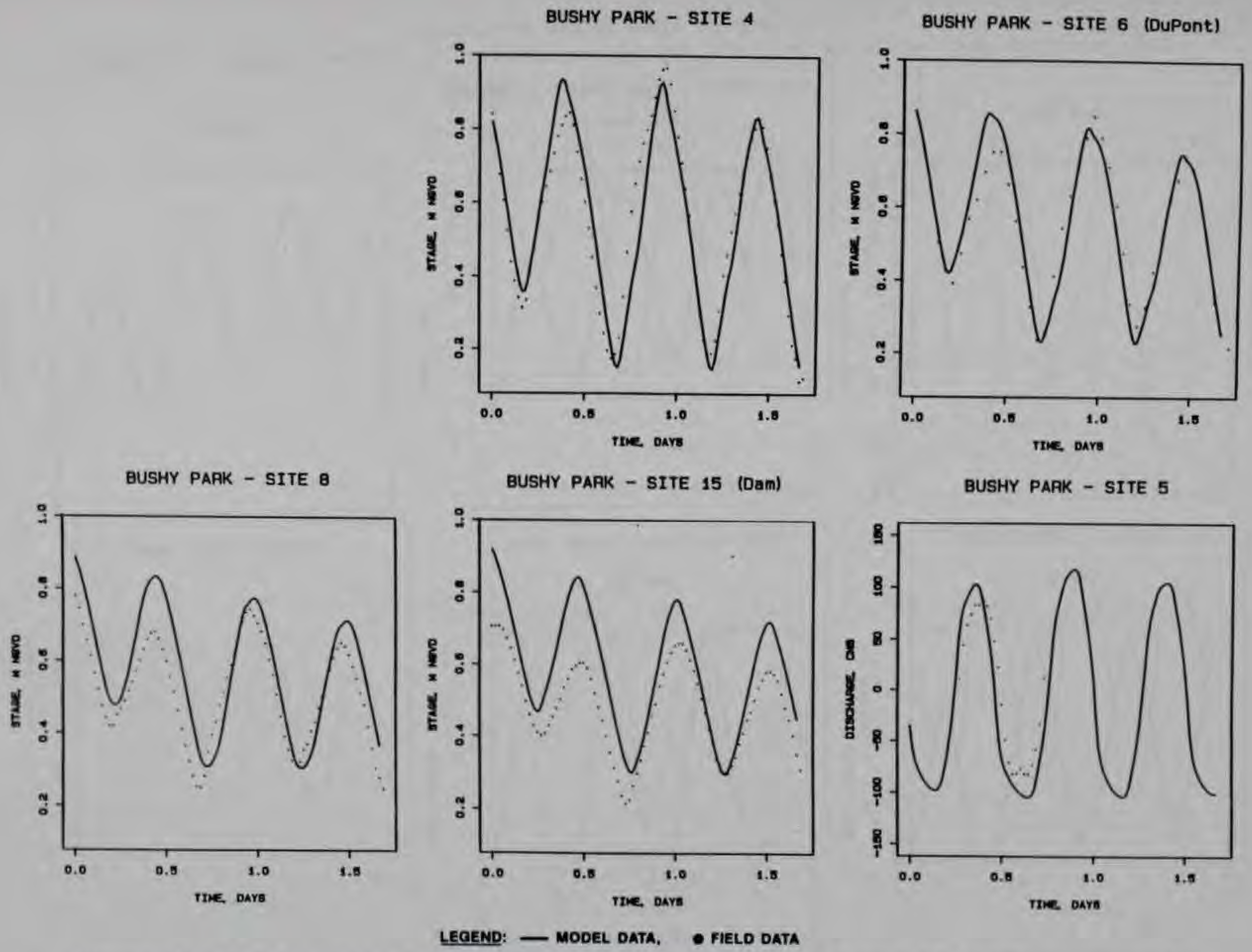


Figure 10. Model results for second data period starting 30 August 1989 with no reservoir withdrawals

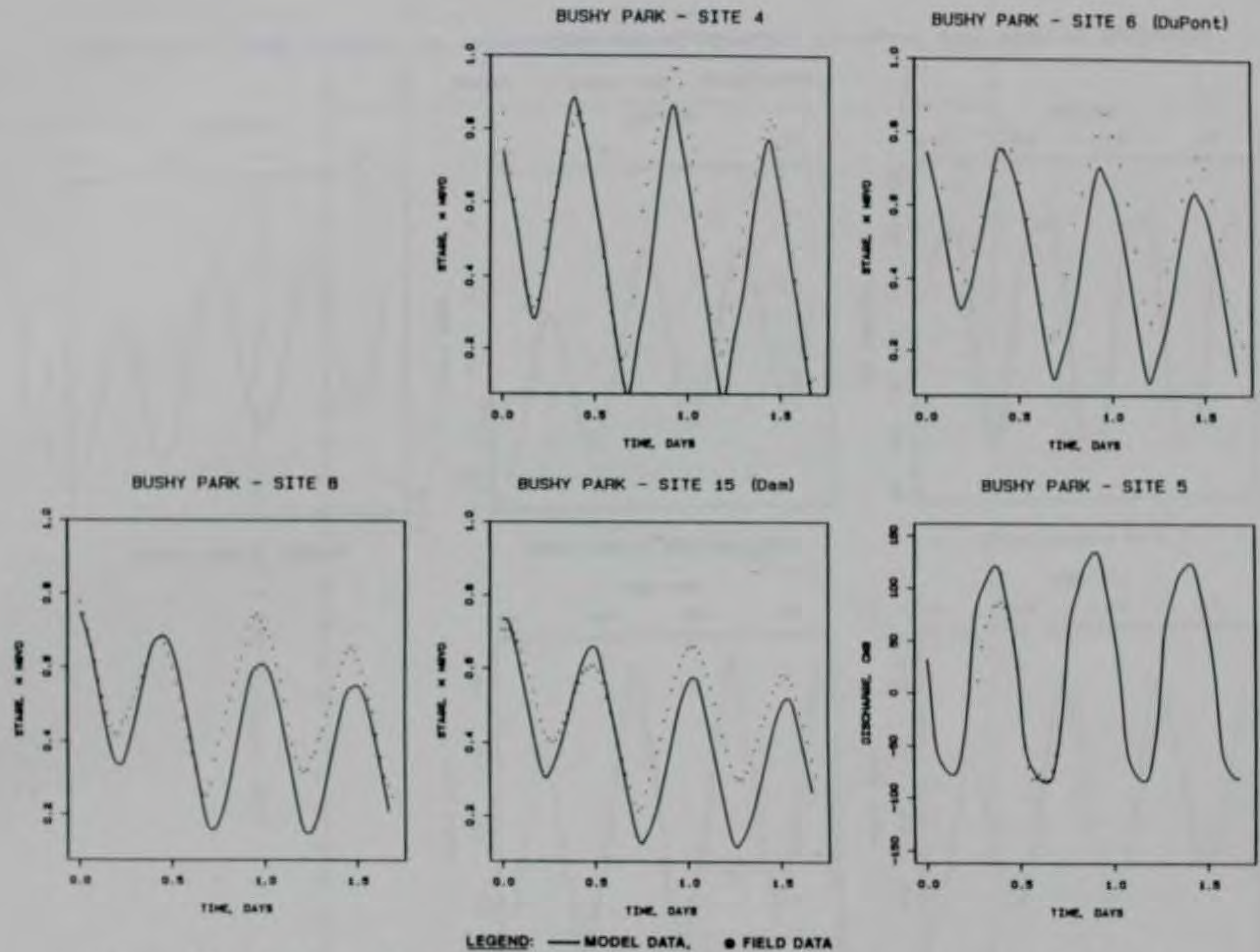


Figure 11. Model results for second data period starting 30 August 1989 with constant reservoir withdrawals of 31 cu m/s

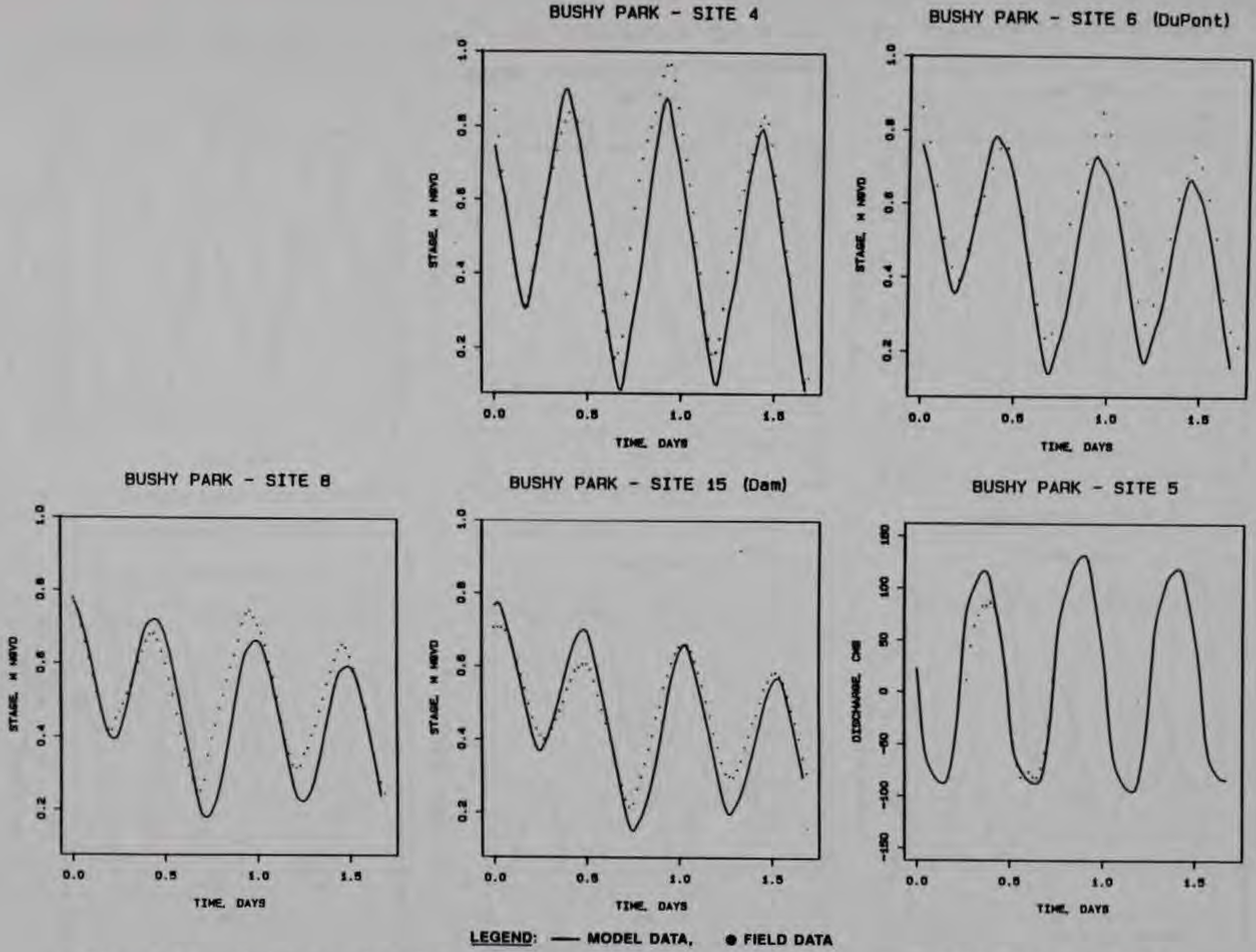


Figure 12. Model results for second data period starting 30 August 1989 with variable withdrawal scenario

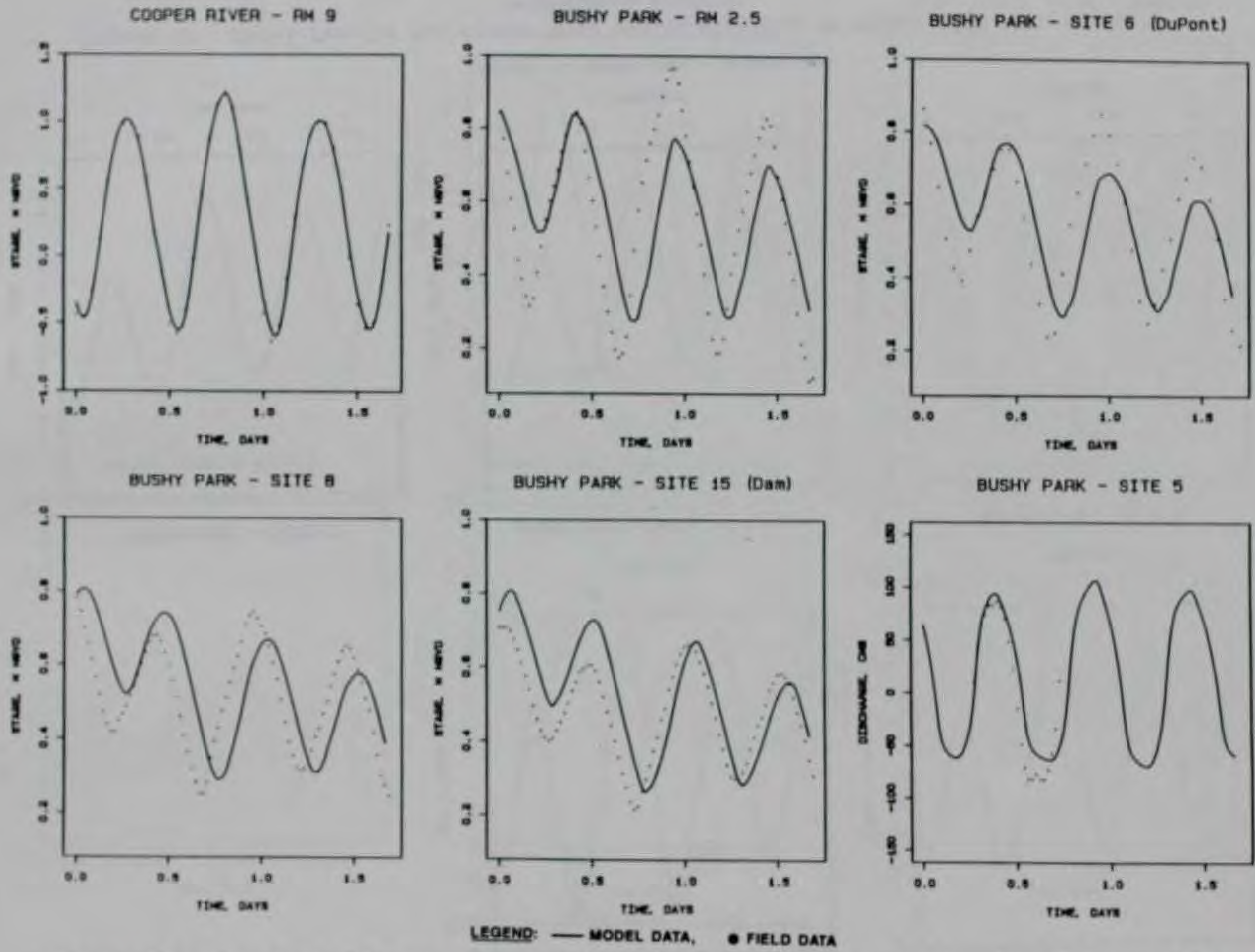


Figure 13. Plan test with present canal dimensions (55 m by 4.2 m), starting 30 August 1989

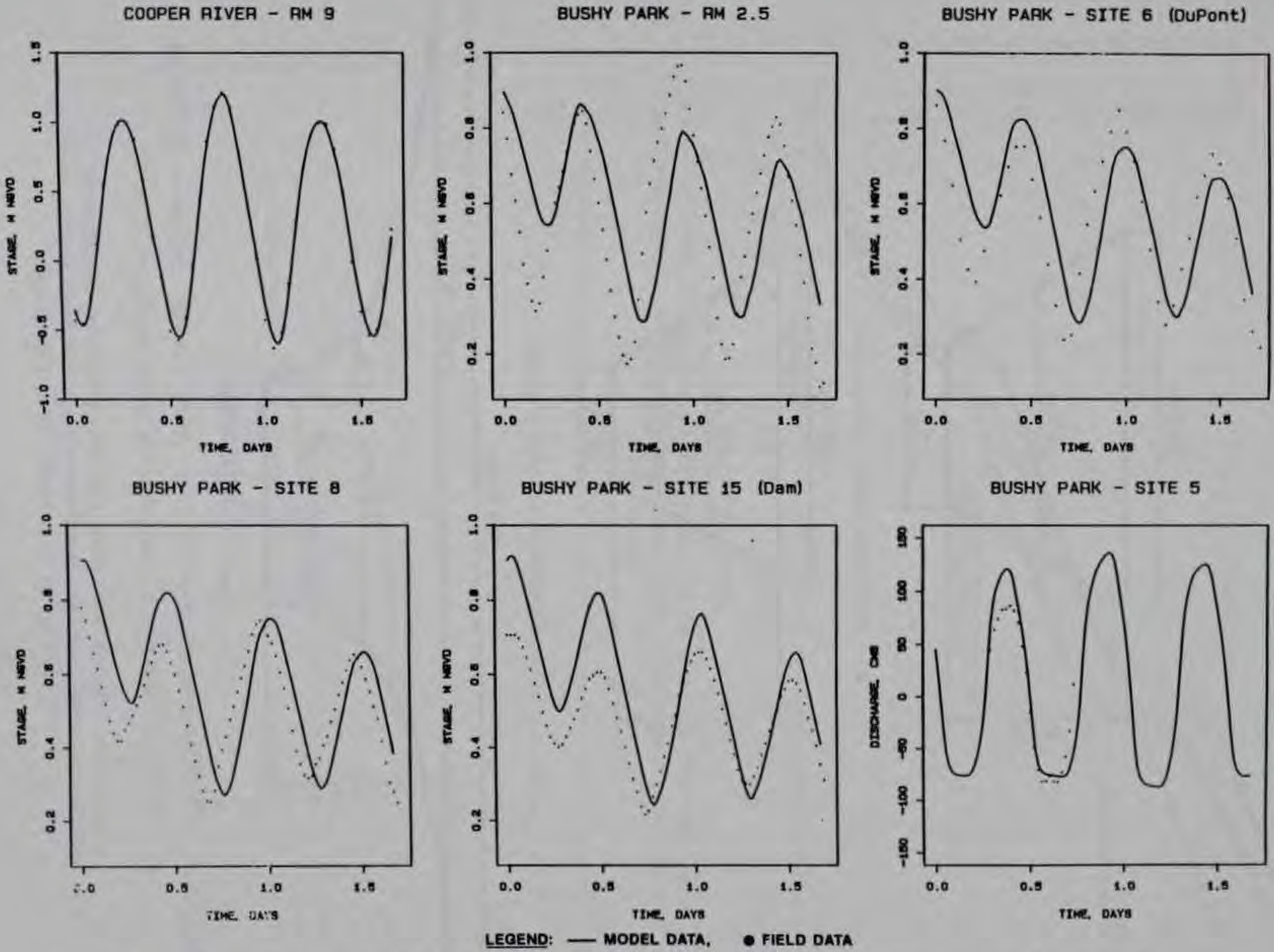
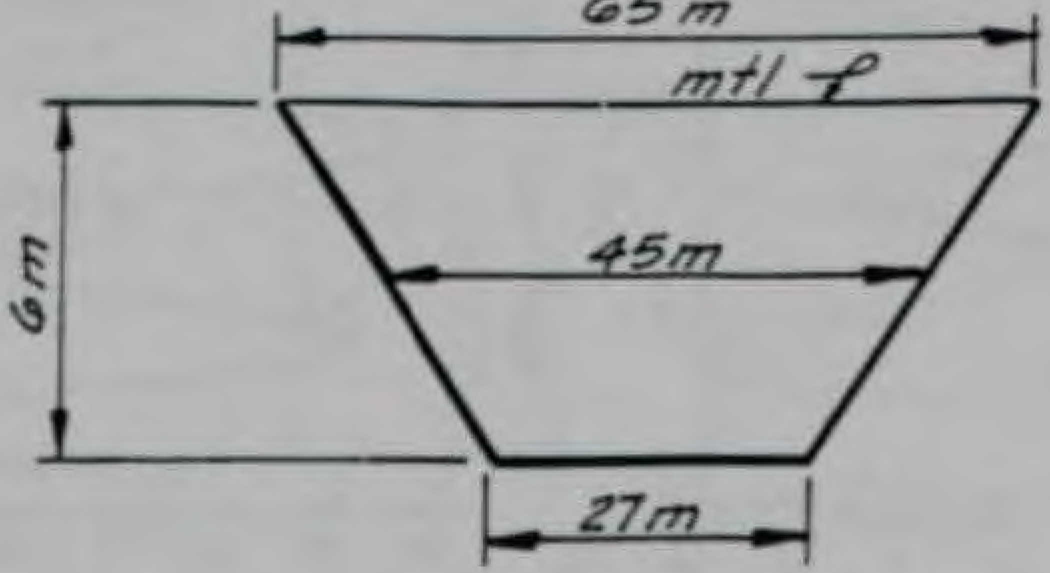
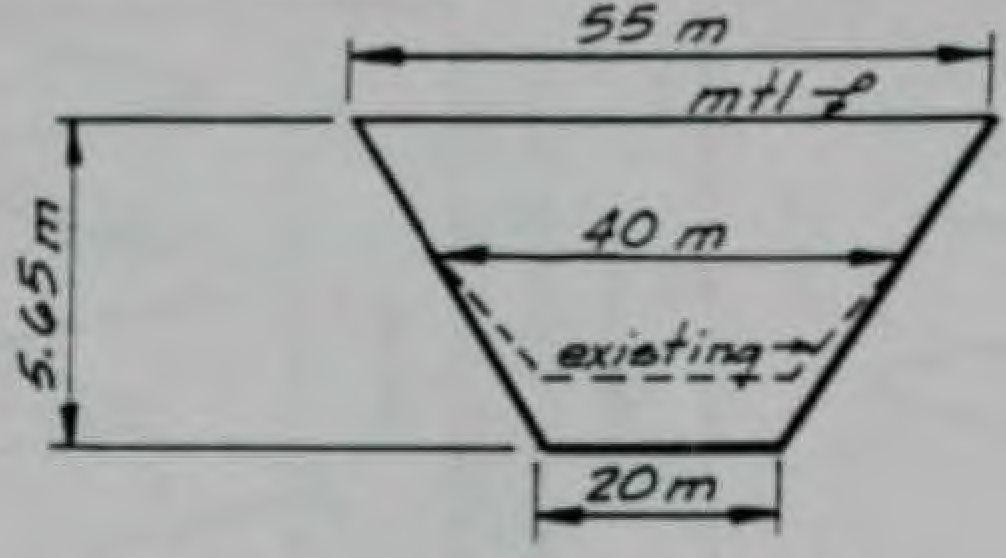


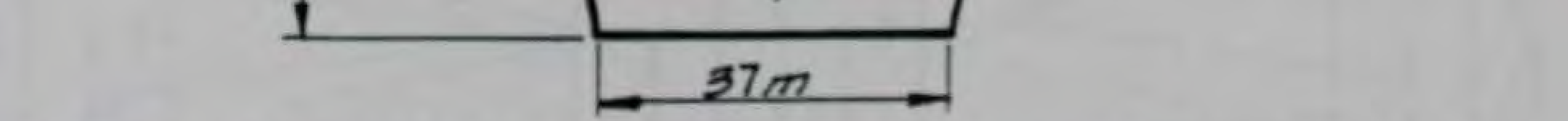
Figure 14. Plan test with 65-m-wide by 6-m-deep lengthened entrance canal, starting 30 August 1989



a. Entrance canal extension



b. Deepened existing canal



NOTE: Difference between mtl and mlw = 0.61 m

c. Deepened reservoir channel (1/2 mile south of site 8)

Figure 15. Cross sections for the proposed lengthened entrance canal design found to maintain tidal amplitude and flushing in the Bushy Park Reservoir

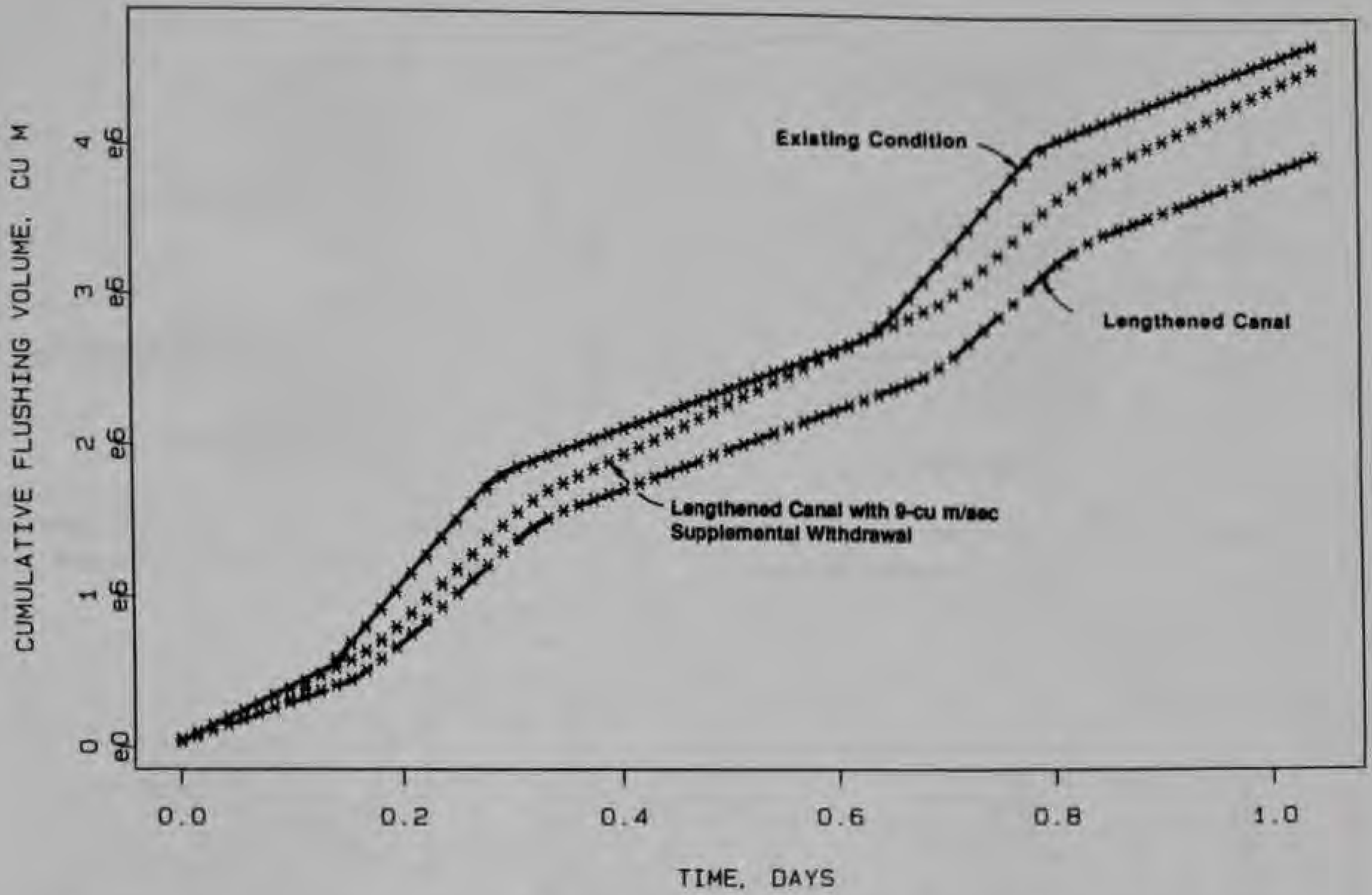


Figure 16. Model cumulative flushing volume results for the base, lengthened canal, and lengthened canal with 9-cu m/sec supplemental withdrawal

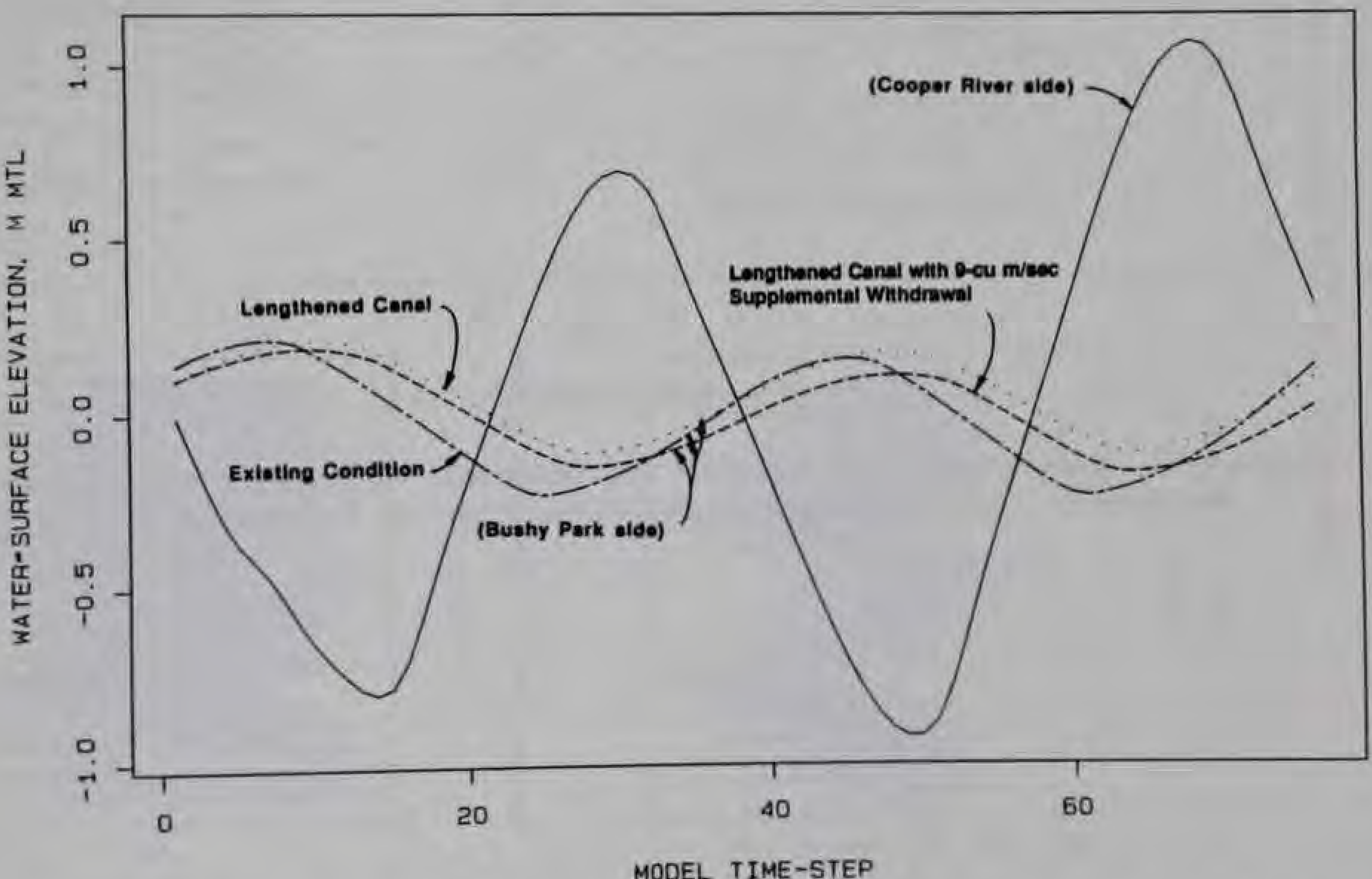


Figure 17. Model water level results for both sides of the Bushy Park Dam for the base, lengthened canal, and lengthened canal with 9-cu m/sec supplemental withdrawal

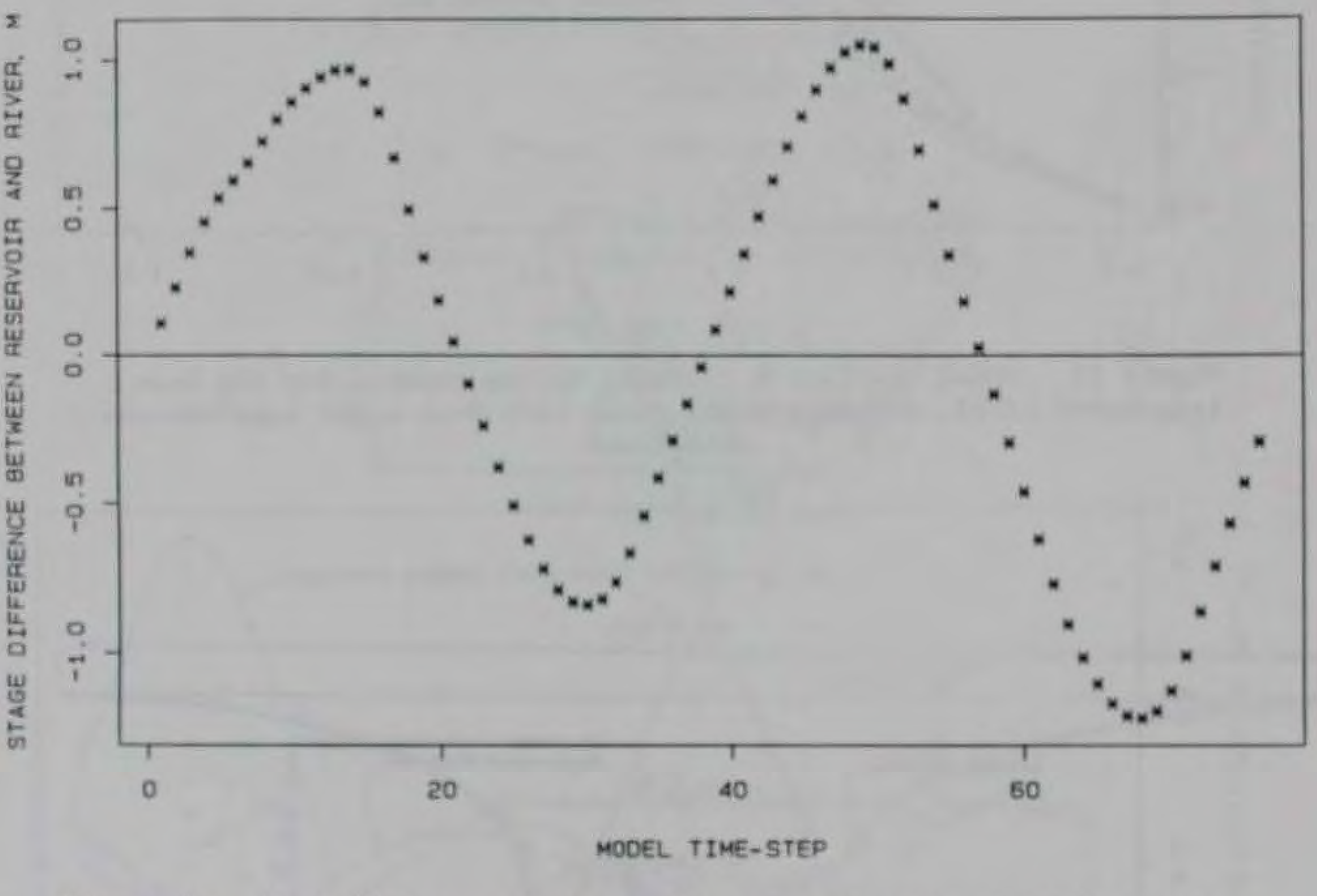
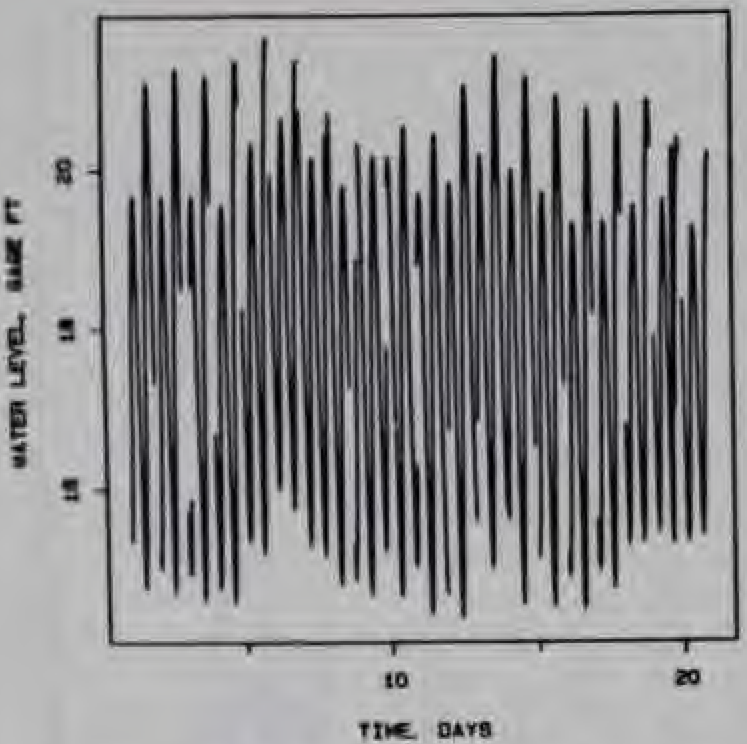
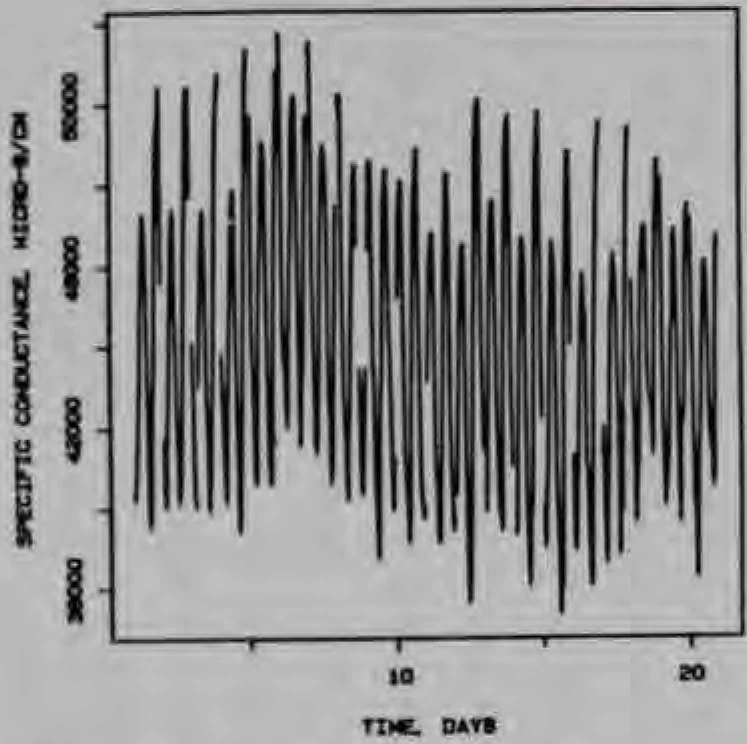


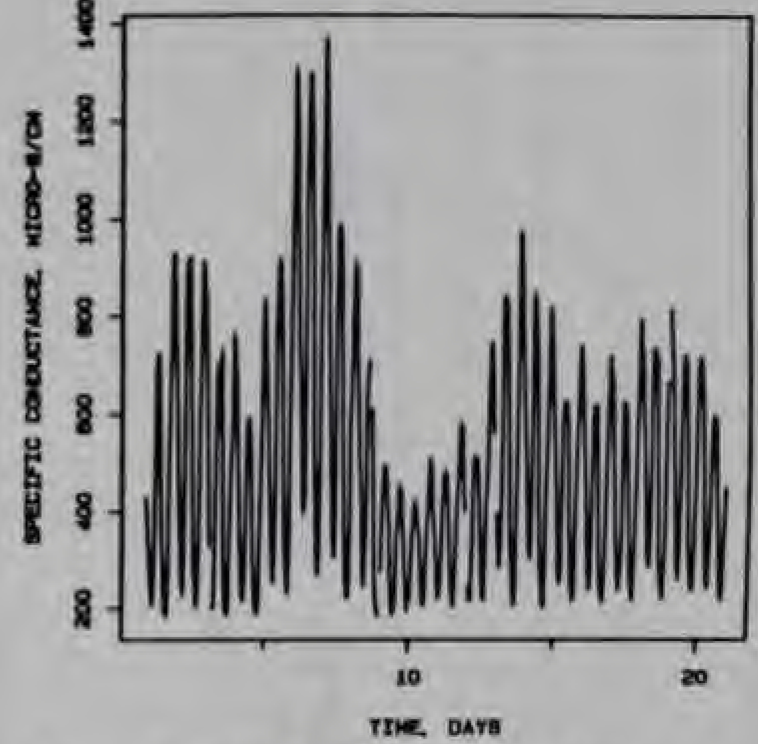
Figure 18. Head difference between the reservoir side (with 9-cu m/sec supplemental withdrawal) and the river side of Bushy Park Dam



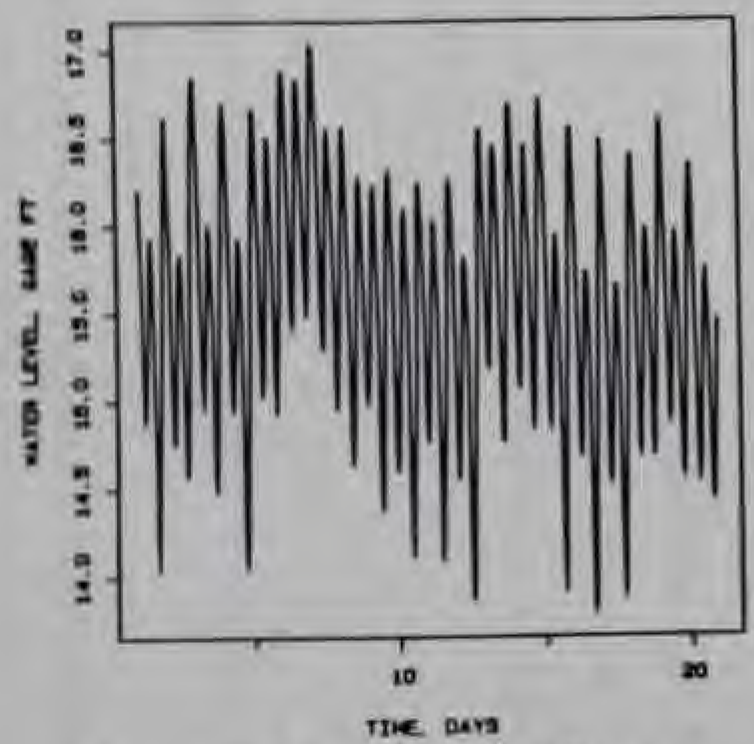
a. Water level, Customs House



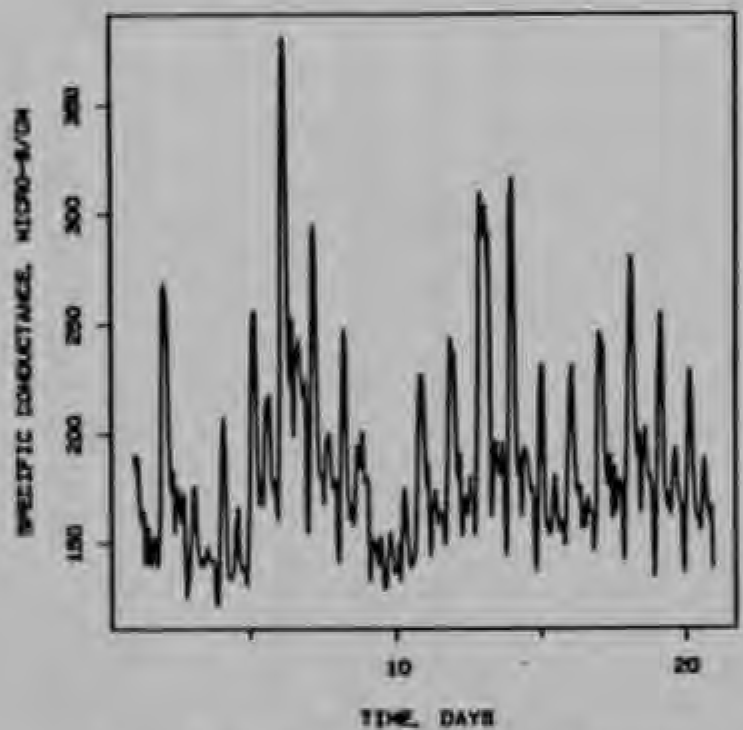
b. Specific conductance, Customs House



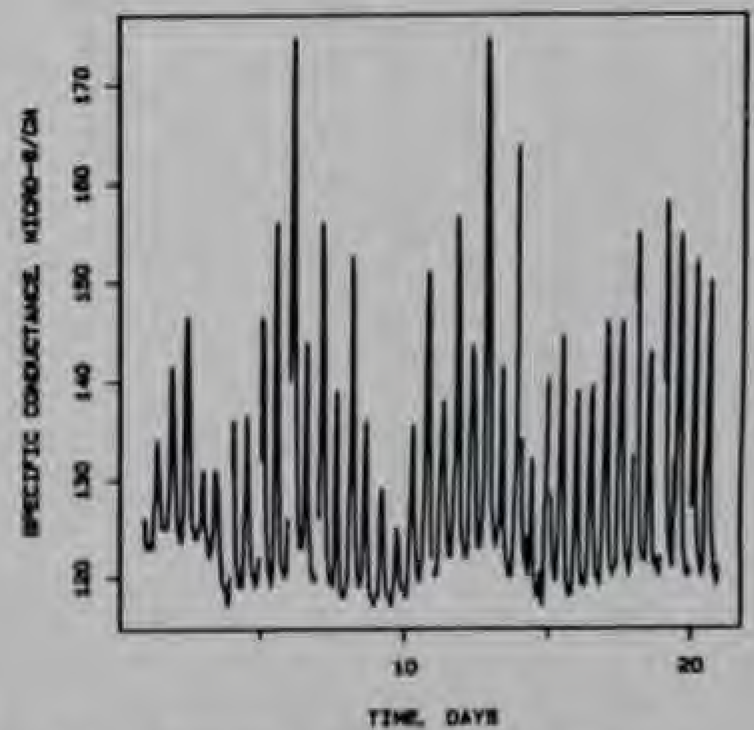
c. Specific conductance, Goose Creek



d. Water level, DuPont



e. Specific conductance, Dupont



f. Specific conductance, Pimlico

Figure 19. Hourly data for the intrusion period starting 30 May 1988

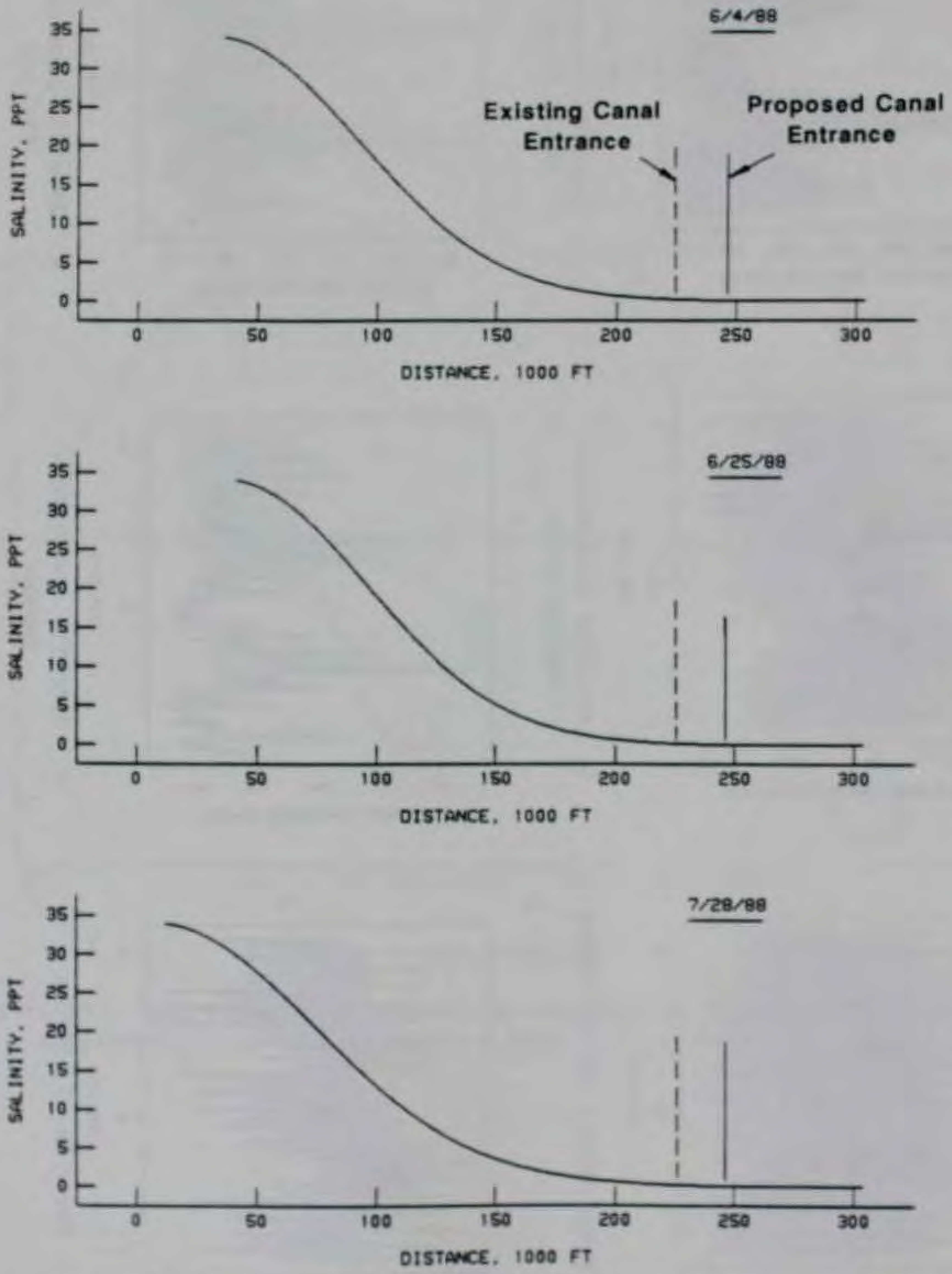


Figure 20. Example model results for three intrusion events

APPENDIX A: NUMERICAL TIDAL MODEL

1. This appendix presents and describes the mathematical equations that make up the flow and salinity portions of the FIBS model used in this study. Due to the complicated nature of this subject, the description will be limited to an overview. Blumberg (1977)* and Wang (1983 and 1984) describe similar laterally averaged flow and salinity models based on the finite difference method.

Equations

2. The following governing laterally averaged dynamic differential equations describe estuarine flow, mixing, and circulation:

- a. The vertically integrated continuity equation.
- b. The horizontal momentum equation.
- c. The continuity equation.
- d. The salinity transport equation.

Additional terms were included in the equations to describe both lateral inflows and tributaries. The following algebraic expressions are required to dynamically modify, or close, the set of differential equations:

- a. The equation of state.
- b. The friction coefficient equation.
- c. The equations for vertical eddy diffusivities for mass and momentum.

An improved vertical eddy diffusivity closure was made using a differential equation for the transport of turbulent kinetic energy.

3. Laterally averaged equations use values averaged across sections at certain increments of depth. The channel widths are specified at each node depth as indicated in the hypothetical channel cross section (Figure A1).

a. Vertically integrated continuity equation:

$$\frac{\partial(B_o h)}{\partial t} + \frac{\partial}{\partial X}\int_{-R}^{h} (UB)\, dZ - \frac{q}{B_o \Delta_s} = 0 \qquad \text{(A1)}$$

* References cited in this appendix can be found in the References at the end of the main body of the report.

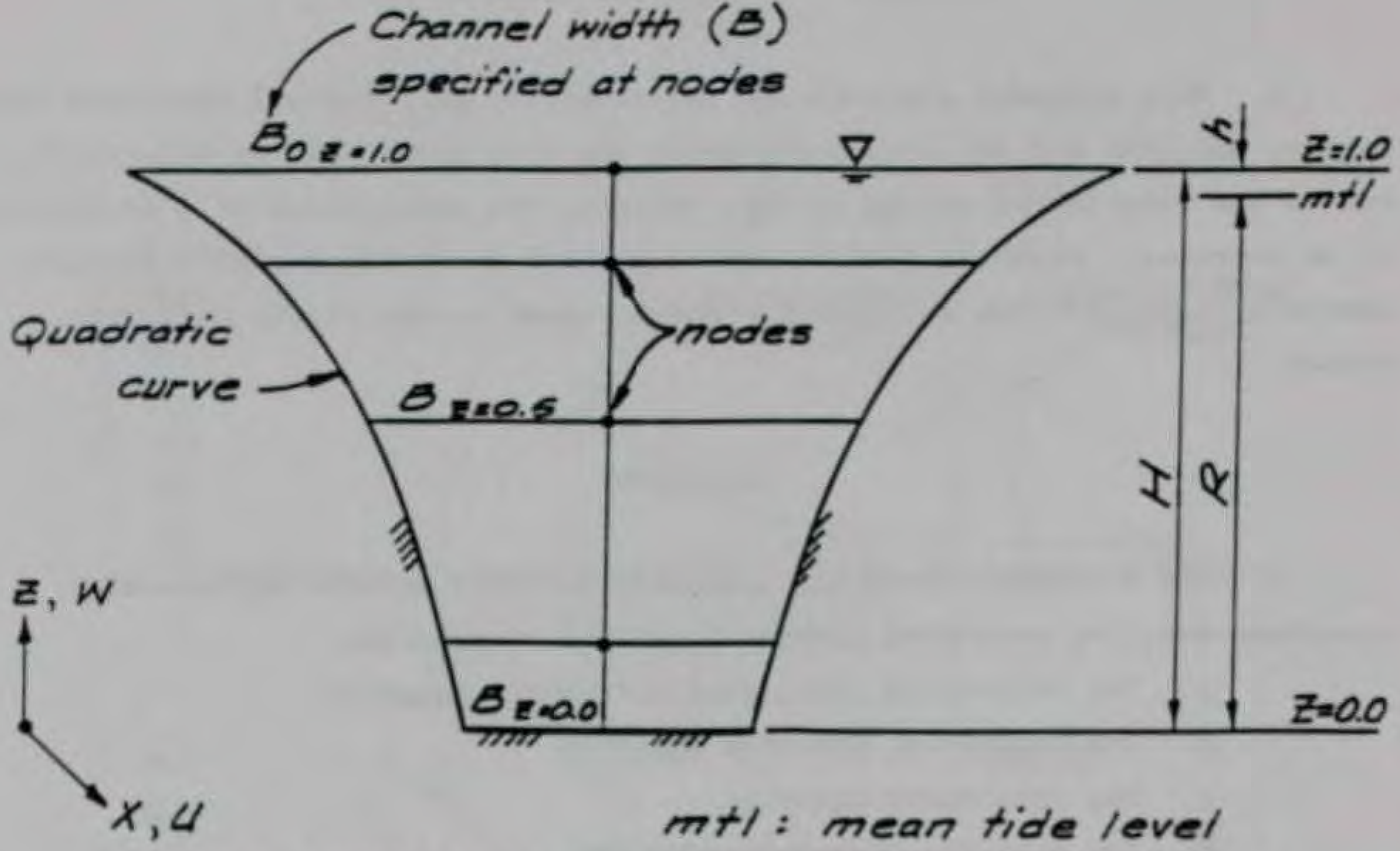


Figure A1. Hypothetical channel cross section (looking seaward)

b. <u>Horizontal momentum equation</u>:

$$\frac{\partial}{\partial t}(UB) + \frac{\partial}{\partial X}(UUB) - BN_x \frac{\partial^2 U}{\partial X^2} + \frac{\partial}{\partial Z}\left[UWB - BN_z \frac{\partial U}{\partial Z}\right] \tag{A2}$$

$$+ C_d U|U| \left|\frac{\partial B}{\partial Z}\right| + gB \frac{\partial h}{\partial X} + g \frac{B}{\rho_o} \int_{z}^{h} \left[\frac{\partial \rho}{\partial X} dZ\right] = 0$$

c. <u>Continuity equation in a vertical plane</u>:

$$\frac{\partial (UB)}{\partial X} + \frac{\partial}{\partial X}(WB) = 0 \tag{A3}$$

d. <u>Salinity transport equation</u>:

$$\frac{\partial (SB)}{\partial t} + \frac{\partial}{\partial X}(SUB) - BK_x \frac{\partial^2 S}{\partial X^2}$$

$$+ \frac{\partial}{\partial Z}\left(SWB - BK_z \frac{\partial S}{\partial Z}\right) - \frac{S, q}{H\Delta_x \int_{z_b}^{h} B\, dZ} = 0 \tag{A4}$$

e. Equation of state:

$$\rho = \rho_o \, (1 + C\phi \cdot S) \tag{A5}$$

where

B_o = breadth at water surface

h = deviation in water surface

t = time

X = horizontal distance

R = depth to mean tide level

U = horizontal (laterally averaged) velocity

B = breadth

Z = vertical distance

q = lateral inflow rate

Δ_e = element length

$N_{x,z}$ = horizontal and vertical eddy diffusivity for momentum

W = vertical (laterally averaged) velocity

C_d = quadratic friction coefficient

g = acceleration due to gravity

ρ_o = freshwater reference density (0.9987 g/cu cm)

S = salinity parameter expressed here as chlorinity, ppm

$K_{x,z}$ = horizontal and vertical eddy diffusivity for mass

ℓ = lateral inflow quantities

H = total depth

$C\phi$ = constant (1.3751 E-6)

f. Bed friction:

$$C_d(x) = \frac{gn^2}{2.22 \, R^{1/3}} \tag{A6}$$

where n is Manning's coefficient of friction.

Vertical eddy diffusivities:

4. Under homogeneous conditions, characteristic eddy diffusivities are $N_{zo} = K_{zo} = K_z'$, where K_z' is an specified constant. The effect of vertical

density stratification is included using the method of Munk and Anderson (1948). Thus:

$$K_z = K_{zo}\,(1 + 3.33\ Ri)^{-(3/2)}$$

and

$$N_z = N_{zo}\,(1 + 10\ Ri)^{-(1/2)}$$

where

$$Ri = \frac{-g}{\rho_o}\,\frac{\partial\rho/\partial Z}{(\partial U/\partial Z)^2}$$

Then at depth,

$$K_z(z) = F_z(z) \cdot K_z \tag{A7}$$

where F_z is a similarity distribution for vertical eddy diffusivity.

Turbulent kinetic energy

5. Instead of assigning a value to K_z', the model will optionally use turbulent kinetic energy κ generated, dissipated, and transported by the flow to calculate vertical diffusivities. The scheme employed is similar to that described by Smith and Takhar (1979) and Smith (1982). Transport is computed in one dimension by

$$\frac{\partial\kappa}{\partial t} + \bar{U}\,\frac{\partial\kappa}{\partial X} - D_x\,\frac{\partial^2\kappa}{\partial X^2} - \frac{\kappa_1 q}{H\Delta_* \int_{\kappa}^{h} B\,dZ} - P_k + \epsilon = 0 \tag{A8}$$

where P_k is production of turbulent kinetic energy

$$P_k = 22.4\left[\frac{U_*^2}{\bar{R}}\right]$$

ϵ is dissipation

$$\epsilon = 1.66\left[\frac{\kappa^{3/2}}{\bar{R}}\right]$$

$\bar{U}$ is the cross-sectional average velocity

$$\bar{U} = \frac{1}{A} \int_{R}^{h} BU \, dz$$

and

D_x = horizontal diffusion coefficient

U_* = shear velocity

A = cross-sectional area

Then, $K_{zo} = N_{zo} = 0.09\ \kappa^{1/2}\ R$, and the values for K_z, N_z, $K_z(z)$ and $N_z(z)$ are computed as previously described.

Boundary Conditions

Ocean boundary

6. In general, the varying tidal conditions are controlled at the ocean boundary of the model mesh. This is accomplished by specifying a tidal water level target value at each time-step in the form of $h'(t) \mid_{X=0} = f(t)$. The model has the capability to synthesize tidal sequences for any specified starting time using 12 constituents and harmonic coefficients provided by the National Ocean Survey. The individual constituents used included M2, S2, K1, O1, P1, N2, L2, Sa, M_sf, Mf, Mm, and Ssa.

7. A nonreflecting ocean boundary was computed for this study by replacing the vertically integrated continuity equation with the following expression:

$$\frac{\partial h}{\partial t} + Cw \frac{\partial h}{\partial X} = -\left[\frac{h(t) - h'(t)}{T_f}\right] \quad \text{(A9)}$$

where

$$Cw = (gH)^{1/2} \mid_{X=0}$$

and T_f is a damping parameter (found to be about 500 sec). This method, similar to that presented by Blumberg and Kantha (1985), allows long waves, or water level perturbations, originating within the model domain to pass through the ocean boundary without reflection. The nonreflecting ocean boundary permits faster stability within the model. In this study, the model reached the stable condition in two tidal cycles. During the flood tidal phases, salinity concentration was specified at the ocean boundary; otherwise the transport boundary was unconstrained.

Upstream boundary

8. Velocities are specified at upstream inflow boundaries and flow rates are specified at lateral inflows. In this study, the condition $U(t) = 0$ was specified at Pinopolis, the upstream end of the model, and a lateral inflow was specified at the adjacent element to represent the hydropower flow release. The Cooper River contributes almost all of the freshwater inflow to the system and is controlled at the Pinopolis Dam. Salinity concentrations (usually zero) were specified at these inflows.

Bottom boundary

9. For this study, the bottom boundary ($Z = 0.0$) was considered to be a slip-flow boundary. The shear stress was imposed by the equation:

$$N_z \frac{\partial U}{\partial Z} = C_d U|U| \tag{A10}$$

Optionally, the model will extrapolate shear stress linearly from within the flow to the bottom boundary using a method suggested by Smith (1982). Within the salinity transport equation, no flux is allowed at the bottom boundary and

$$US + K_z \frac{\partial S}{\partial Z} = 0 \tag{A11}$$

Vertically Stretching Coordinates

10. In the vertical or Z-direction, computational nodes are assigned to the surface, intermediate, and bottom levels. When changes occur in the water levels, all nodes except the bottom node move, or stretch, to conform with the

depth of flow. The typical configuration of the nodes within an element are shown in Figure A2. The X and Z are the actual world coordinates. The x and z are the nondimensional element coordinates that vary in each element

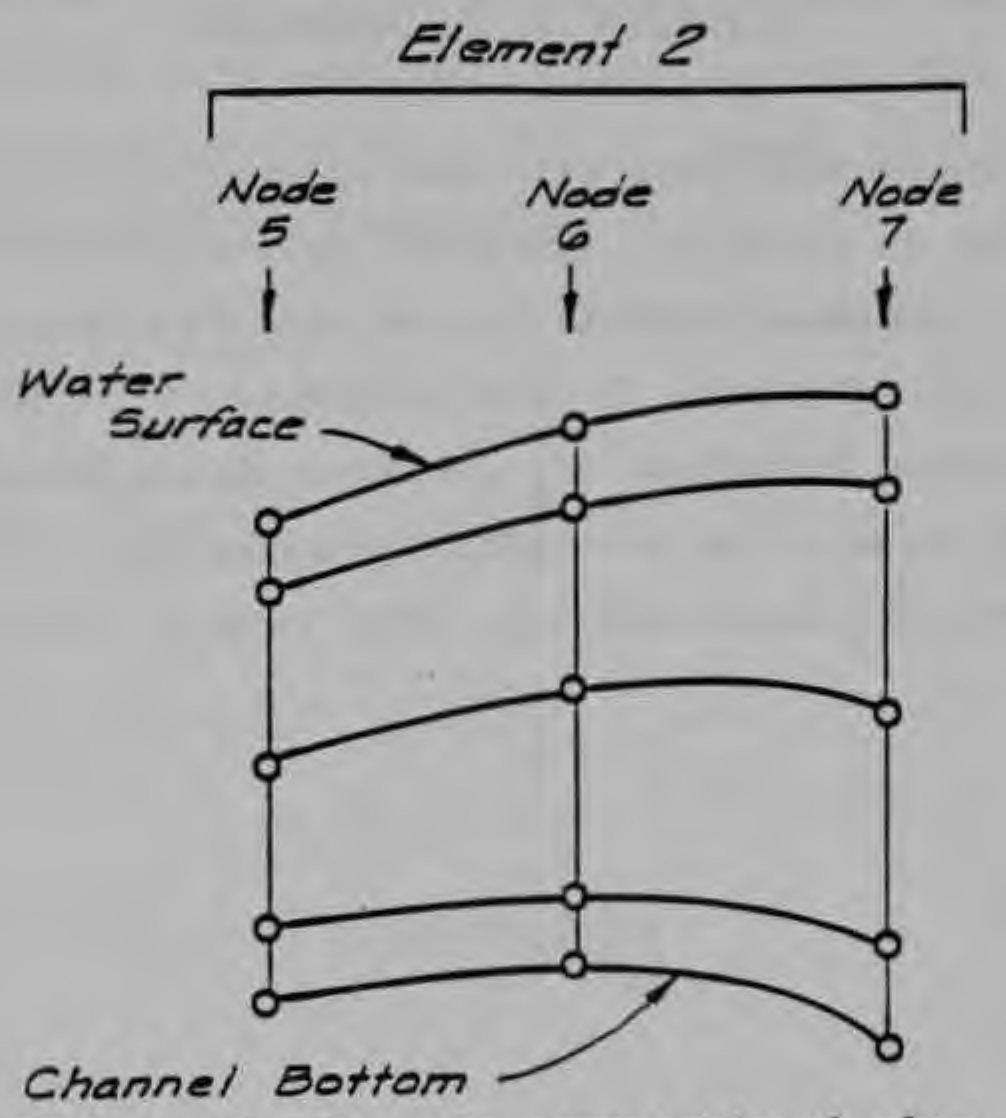


Figure A2. Example computational element from schematic mesh showing stretching coordinates and nodes

from 0.0 to 1.0 upstream and vertically upward, respectively. The two coordinate systems are related by

$$\frac{\partial}{\partial Z} = \frac{1}{H} \frac{\partial}{\partial z} \tag{A12}$$

$$\frac{\partial}{\partial X} = \frac{1}{\Delta_e} \frac{\partial}{\partial x} \tag{A13}$$

The stretching coordinates system introduces new terms into the governing differential equations. A derivative in some level of the X-direction is calculated by the following expression:

$$\frac{\partial}{\partial X} = \frac{\partial}{\Delta_e \partial x} - \left(\frac{Z}{R\Delta_e} \frac{\partial h}{\partial x} \frac{\partial}{\partial Z} + \frac{(1 - z)}{R\Delta_e} \frac{\partial R}{\partial x} \right) \frac{\partial}{\partial Z} \tag{A14}$$

This method is similar to the finite-difference methods presented by Sheng (1983).

Solutions of the Equations

11. The governing equations were solved over the previously described finite elements using an explicit orthogonal collocation method of weighted residuals. Weighted residual methods can be used to expand an unknown solution in a set of trial functions T over a domain v. The collocation method applies weighting functions (w_j, a Dirac delta function) to the trial functions, chosen to be a set of orthogonal polynomials. The node locations x_j satisfy the roots of the polynomials. The form of the collocation method is now

$$\int_v w_j T \, dv = T \Big|_{x_j} \tag{A15}$$

where $w_j = \delta(x - x_j)$.

12. Instead of solving for the unknown trial function coefficients, a major simplification described by Finlayson (1972) is that the solution can be derived in terms of values at the collocation nodes (r). The nodes are located at Gauss points for this purpose, and the method is known as orthogonal collocation. The derivatives and integrals can therefore be expressed in terms of values of the function at the collocation nodes by

$$\frac{dr}{dx} = A\,r \quad \text{and} \quad \frac{d^2r}{dx^2} = B\,r \tag{A16}$$

The matrices A and B are computed from the trial functions.

Integration in Time

13. An explicit time integration method was used to compute variables

dynamically over time. The fourth-order Runge-Kutta method was used in this study. This method is very accurate, and often used to solve "stiff" equation sets, equations with distinct stability limits. Results were found to be independent of time-step, Δt, up to a Courant number, $\Delta t/\Delta_e \cdot Cw$, of 1.0 (or 2.0 based on longitudinal node spacing). Starting with the initial conditions y_n at time t_n, the function $y'(t) = f(t,y)$ where y' is the rate of change in y is solved in four steps over Δt :

a. $K_0 = \Delta t \cdot f\,(t_n, y_n)$.

b. $K_1 = \Delta t \cdot f \left[t_n + \frac{\Delta t}{2} , y_n + \frac{K_0}{2}\right]$.

c. $K_2 = \Delta t \cdot f \left[t_n + \frac{\Delta t}{2} , y_n + \frac{K_1}{2}\right]$.

d. $K_3 = \Delta t \cdot f \left[t_n + \Delta t , y_n + K_2\right]$.

Finally, the result of the four steps are combined to complete the time-step:

$$y_{n+1} = y_n + \frac{1}{6}\,(K_0 + 2K_1 + 2K_2 + K_3) \qquad \text{(A17)}$$

Waterways Experiment Station Cataloging-in-Publication Data

Teeter, Allen M.
Effects of a proposed Bushy Park entrance canal relocation, Cooper River, South Carolina / by Allen M. Teeter ; prepared for U.S. Army Engineer District, Charleston.
58 p. : ill. ; 28 cm. — (Technical report ; HL-92-8)
Includes bibliographic references.
1. Cooper River (S.C.) — Channelization — Environmental aspects. 2. Saltwater encroachment — South Carolina — Bushy Park Reservoir. 3. Hydraulics — Mathematical models. 4. Bushy Park Reservoir (S.C.) I. United States. Army. Corps of Engineers. Charleston District. II. U.S. Army Engineer Waterways Experiment Station. III. Title. IV. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-92-8.
TA7 W34 no.HL-92-8