RETENTION TIME SIMULATION FOR BUSHY PARK RESERVOIR NEAR CHARLESTON, SOUTH CAROLINA

By David E. Bower, Curtis L. Sanders, Jr., and Paul A. Conrads

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CONTENTS

ILLUSTRATIONS

TABLES

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ABSTRACT

Several scenarios were used to evaluate the effectiveness in reducing the retention time for water in Bushy Park Reservoir near Charleston, S.C. Flows were simulated by using the U.S. Geological Survey BRANCH one-dimensional unsteady-flow model on the Cooper River from Pinopolis Dam at Lake Moultrie to Yellow House Creek, 5 miles seaward of Back River, and on the Bushy Park Reservoir. Flushing of Bushy Park Reservoir was simulated by using the particle-tracking function of the BRANCH model, which accounts only for hydrodynamic movement without diffusion, dispersion, or decay of contaminants. The model was calibrated and verified by using data from 15 flow-measurement sites and 17 stage stations.

Results are quantified on graphs showing the number of days required to move a particle of water (retention time or days-to-flush) from Durham Canal to the Charleston Commissioners of Public Works (CPW) intake, reflecting changes in the amount of withdrawal by Charleston CPW; the location of the CPW intake at its present location on Foster Creek or at a new location 0.9 mile north of Foster Creek, on the Back River; flow through hypothetical flap-type tide gates at (6-foot concrete pipes) Bushy Park Dam; and whether a thermoelectric power plant is withdrawing water from the reservoir.

The withdrawals by the thermoelectric power plant are large enough to improve the quality of water in the reservoir from Durham Canal to its intake to a degree that the water can be economically treated by CPW. Target flushing rates of 3.1 and 5.2 days were established for hypothetical CPW intakes located 0.9 mile north of Foster Creek on the Back River and at the current Foster Creek intake, respectively. This flushing rate uses the same flushing rate achieved by the power plant. Combined maximum CPW withdrawals from the Edisto River and Foster Creek are 50, 118, and 150 million gallons per day (Mgal/d) for current, short-term, and ultimate demand projections, respectively. With 50 Mgal/d withdrawals at Foster Creek, the days-to-flush is about twice the estimated target for eight 6-foot concrete pipes. If the withdrawal rate is increased to 118 or 150 Mgal/d for the same number of pipes, the target rate is exceeded by one day. If the CPW intake is moved to the site on the Back River, the target days-to-flush can be reached by withdrawals of 50 to 150 Mgal/d with six to eight 6-foot concrete pipes. Significant improvement in flushing characteristics could be achieved if the intake was located on the Back River, 0.9 mile north of Foster Creek. A sensitivity analysis showed that flushing rates were insensitive to model roughness estimates, cross-section datums, or boundary-condition stage datums.

INTRODUCTION

The Charleston Commissioners of Public Works (CPW) provides drinking water from two sources of freshwater for about 400,000 people west of the Cooper River in the vicinity of Charleston, S.C. From 65 to 90 percent of the water supply is withdrawn from the Edisto River near Givhans, S.C., and from 10 to 35 percent is withdrawn at an intake on the Foster Creek part of the Bushy Park Reservoir at site W1 (fig. 1). The Foster Creek intake is used during periods of high-water usage and when the supply from the Edisto River is inadequate. The reservoir is formed by an earthen dam with no outlet across the Back River and is supplied by Durham Canal, which connects it to the West Branch Cooper River (fig. 1). The reservoir also includes the Upper Back River and Foster Creek. Several industries withdraw as much as 560 Mgal/d, which is equivalent to a daily withdrawal of 866 ft^3/s, from the reservoir.

Bushy Park Reservoir and its tributaries are eutrophic and contain large amounts of aquatic vegetation. Jordon, Jones, and Goulding (1988) determined that water in the reservoir at times does not meet State standards for dissolved oxygen, that concentrations of organic compounds at times cause taste and odor problems, and that fecal coliform counts are higher in Foster Creek than in the reservoir.

In May 1990, the U.S. Geological Survey (USGS) in cooperation with the Charleston CPW, initiated a study to determine the effectiveness of selected scenarios of withdrawals and operation of hypothetical tide gates at Bushy Park Dam in reducing retention time of water in Bushy Park Reservoir.

Purpose and Scope

This report describes the results of a study to determine simulated flows using the USGS's one-dimensional (1-D) unsteady-flow model BRANCH (Schaffranek and others, 1981) on the Cooper River from Pinopolis Dam to Yellow House Creek (fig. 1) and on the Bushy Park Reservoir. Flushing of Bushy Park Reservoir was simulated by using the particle-tracking function of the BRANCH model. Particle tracking only accounts for hydrodynamic movement of water particles and does not account for dispersion, diffusion, or decay of contaminants; however, the particle-tracking method adequately defined retention times of water in the reservoir.

Study Area

The study area is located in Berkeley County, South Carolina, within the lower Coastal Plain physiographic province. The model network includes the Cooper River from the "Tee" to its confluence with Yellow House Creek, the Lake Moultrie Tailrace Canal, the West Branch Cooper River from the Tailrace Canal to the "Tee," part of the East Branch Cooper River, and Grove Creek, Flag Creek, Yellow House Creek, Durham Canal, and Bushy Park Reservoir, which includes Foster Creek, Back River, and Upper Back River (fig. 1).

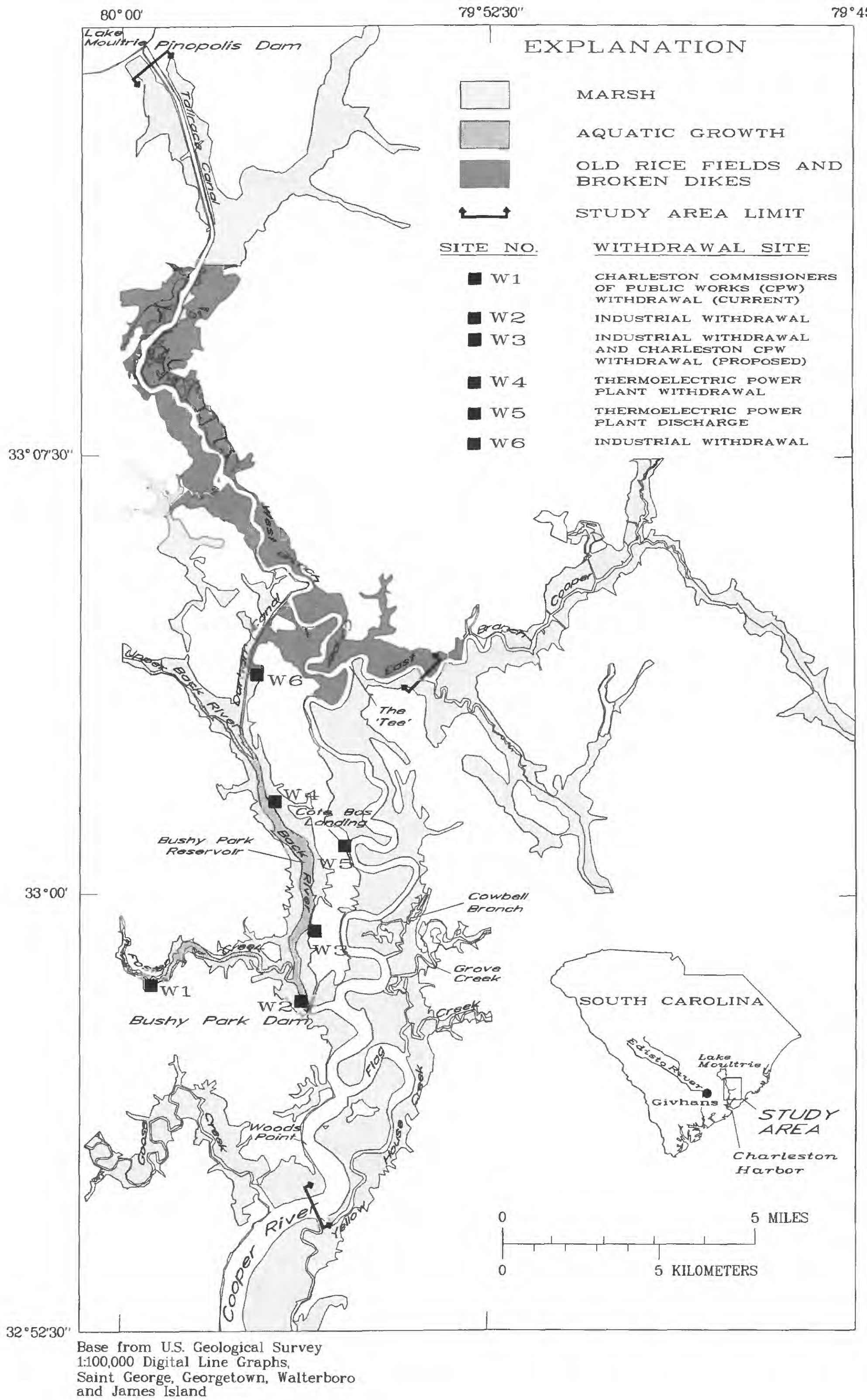


Figure 1.--Study area and locations of withdrawal and discharge sites.

Most of the flow in the Cooper River is released from Pinopolis Dam into the Tailrace Canal (fig. 1). The channel downstream to the confluence with Flag Creek is mostly natural, and is surrounded by tidal marshes and old rice fields in varying states of disrepair. This area contains large amounts of poorly defined overbank storage and unmeasurable flows through broken levees between the channel and the rice fields.

Downstream of Flag Creek, the main channel is maintained by the U.S. Army Corps of Engineers for navigation of ocean-going vessels. The tidal marshes and tributaries in this area are mostly natural.

The major natural tributaries to the Bushy Park Reservoir are the Upper Back River (upstream of the confluence with Durham Canal) and Foster Creek, which contain approximately 2,500 acres of swampy areas. Most of the flow into the Bushy Park Reservoir, however, comes from Durham Canal, which connects the West Branch of the Cooper River to the reservoir. The canal is approximately 3 mi long, 150 ft wide, and 17 ft deep. The canal and Bushy Park Dam were built in 1955 and 1956 by the Bushy Park Authority to form a convenient freshwater reservoir for industrial and municipal use. The Charleston CPW purchased the assets of the Bushy Park Authority in 1964 and controls use of the waters from the reservoir. The Back River part of Bushy Park Reservoir has a length of approximately 5.5 miles, with widths ranging from 690 to 2,200 ft and depths ranging from 12 to 45 ft.

Foster Creek has low dissolved-oxygen concentrations, occasionally high concentrations of organic compounds causing objectionable taste and odor, and occasionally high fecal coliform counts (Jordon, Jones, and Goulding, 1988). The reservoir is eutrophic and is heavily vegetated with aquatic plants that thrive only in freshwater, such as water hyacinth, water primrose, and hydrilla. The South Carolina Water Resources Commission routinely applies herbicides to the aquatic growth, requiring periodic interruption of municipal and industrial withdrawals.

Pinopolis Dam was constructed in 1941 to form Lake Moultrie (fig. 1) for the purpose of generating electricity from water diverted from the Santee River. Daily mean flows in the Cooper River were increased from 200 to about 15,000 ft^3/s with extremes as much as 33,700 ft^3/s. Because the increased flows increased the sedimentation rate in Charleston Harbor, flows were rediverted from Lake Moultrie to the Santee River in 1985. Since this rediversion, average flows of about 4,500 ft^3/s (Bennett and others, 1989 and 1990) have been maintained from Pinopolis Dam, with releases as necessary (as much as 23,900 ft^3/s) to keep brackish water from moving upstream into Durham Canal.

The Charleston CPW withdraws as much as 91 ft^3/s from Foster Creek in comparison to industrial withdrawals of as much as 6.88, 7, 869, and 3.11 ft^3/s from the Back River part of Bushy Park Reservoir at sites W2, W3, W4, and W6 (fig. 1), respectively. Except for the Foster Creek withdrawal, all withdrawals are immediately released into the Cooper River upstream of Back River. The intake for the Charleston CPW on Foster Creek (site W1) is identified as the "Foster Creek intake" in this report.

Previous Studies

The Cooper River and Charleston Harbor are valuable assets to South Carolina. Several sediment, water-quality, and flow studies have been conducted on the Cooper River and Charleston Harbor because of intense industrial, municipal, power generation, and navigational uses of the waters.

The U.S. Army Corps of Engineers, Charleston District, (1966), determined that the shoaling of Charleston Harbor was caused by the diversion of water from the Santee River (South Carolina Water Resources Commission, 1979) to the Cooper River for power generation in 1942.

Lagman and others (1980) investigated the water-quality and aquatic-vegetation conditions of Bushy Park Reservoir and detected elevated concentrations of chloride and plant nutrients (nitrogen and phosphorous) in Foster Creek. Water with high chloride concentrations may not be usable by municipalities and industries. Nitrogen and phosphorous contribute to the growth of nuisance aquatic plants.

A low-flow investigation of Charleston Harbor, which included salinity simulations (Chigges, 1981), was conducted by using the Charleston harbor dynamic estuary model. Patterson (1983) studied the effects of the proposed Cooper River rediversion on sediment in Charleston Harbor. He noted that it may take 10 years for the rediversion effects to stabilize.

Bushy Park Reservoir and its tributaries are eutrophic and contain large amounts of aquatic vegetation. Jordon, Jones, and Goulding (1988) determined that water in the reservoir at times does not meet State standards for dissolved oxygen, that concentrations of organic compounds at times cause taste and odor problems, and that fecal coliform counts are higher in Foster Creek than in the reservoir. That verified earlier studies of the effects of aquatic vegetation, especially in Foster Creek and the Bushy Park Reservoir. Specific contaminants, such as 2-methylisoborneol and geosmin, which cause taste and odor problems when used for human consumption, were identified.

Teeter (1989) showed the effects of Cooper River rediversion flows on shoaling conditions in Charleston Harbor. Teeter and Pantow (1989) used a schematic numerical model to determine the effects of harbor deepening for navigation on sedimentation in Charleston Harbor.

By 1989, the aquatic plant hydrilla had infested the upper part of Foster Creek in the vicinity of Foster Creek intake and treatment with an herbicide had become routine. The herbicide may be used in potable water supplies provided application is made at least 0.25 mi away from all potable water intakes (oral commun., Larry Lagman, South Carolina Water Resources Commission, March 1992). The herbicide requires a relatively long contact time to be effective. Therefore, de Kozlowski (1990) conducted a dye study in Foster Creek near the Foster Creek intake to determine flow paths that might be taken by the herbicide. Because Foster Creek is tidally affected, dye was injected 0.25 mi upstream and 0.25 mi downstream of the Foster Creek intake. After five days, the dye had not reached the mouth of Foster Creek.

The latest study (Teeter, 1992) determined the effects of relocating the northern entrance of Durham Canal to a point closer to Pinopolis Dam. The study indicates that the relocation would maintain the present flushing capacity of the reservoir and would give additional safeguards against brackish water getting into Bushy Park Reservoir.

DATA COLLECTION

To provide the data to calibrate and run the BRANCH model, stage data were collected at 23 gaging stations. Two sets of flow measurements were made by USGS personnel over a part of a tidal cycle at 15 selected locations. Cross sections were determined for 117 locations to define more than 73 mi of channel geometry. Records of water withdrawals from the Bushy Park Reservoir were obtained from the industries located at sites W2, W3, W4, and W6 as shown in figure 1, and from the Charleston CPW (W1).

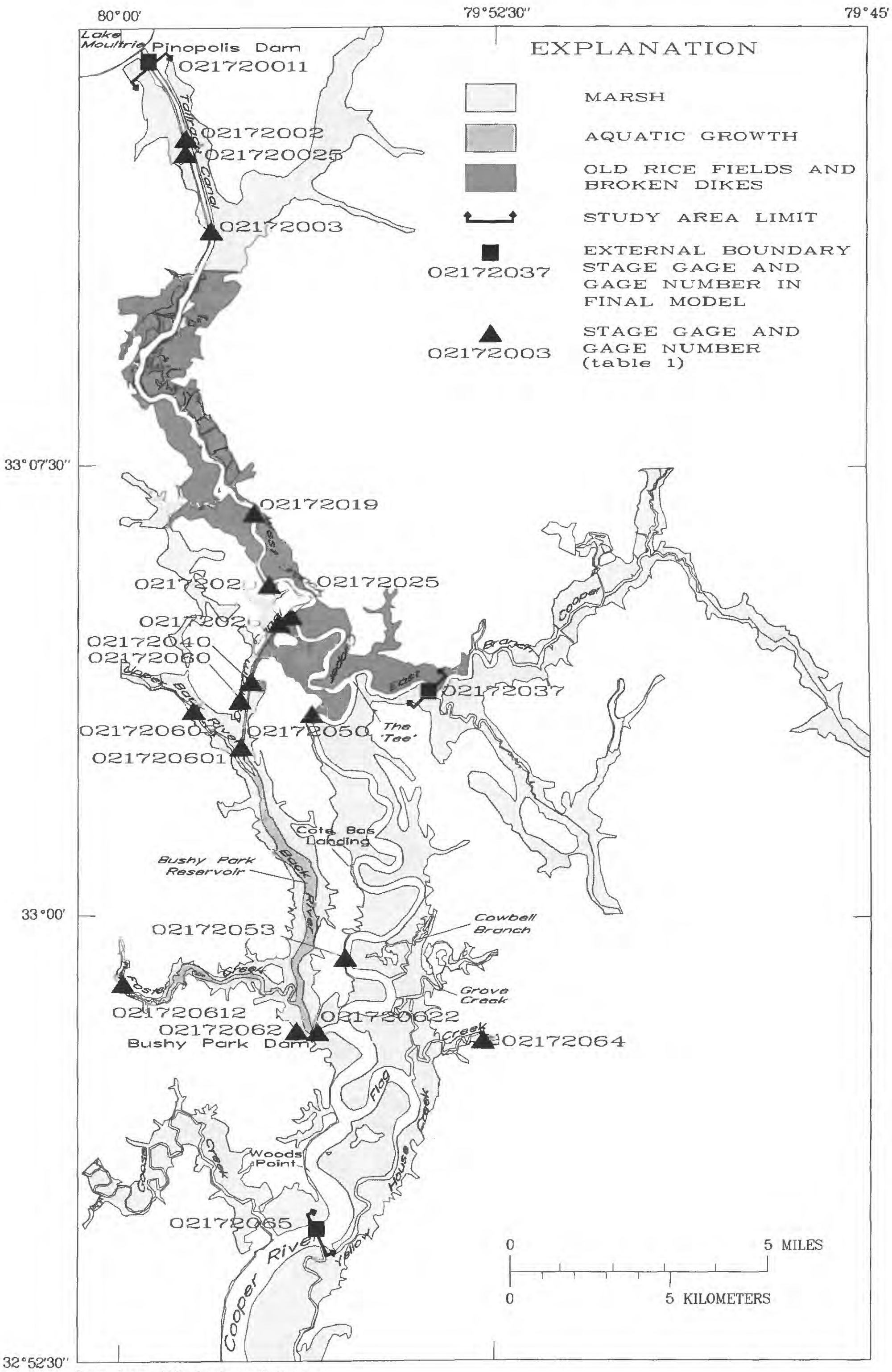
Stage Data

Stage data were collected by using automatic digital recorders and satellite data-collection platforms at stilling-well type gages. Datums were established by standard levels, electronic distance measuring (EDM) equipment, and global positioning system (GPS) equipment. Daily mean stages over several days of concurrent record at all gages were compared to detect gross errors in datum on the assumption that daily mean elevations should be nearly equal in a tidal environment.

Locations of stage gages are shown in figure 2 and listed in table 1. Stages at stations 021720011, 02172037, and 02172065 were part of the external boundaries used to drive the final flow model. Zero flows were used as external boundaries at the upper ends of Foster Creek, Upper Back River, Grove Creek, and Flag Creek. Stages at stations 02172025, 021720603, 021720612, and 02172064 were also used as external boundaries during the calibration process when the study area was divided into three separate models. Stage data from internal continuous-record stations were used for model calibration and verification.

Flow Data

Flows were measured from boats and bridges at 15 locations in the study area to characterize the hydrodynamics for the flow model (fig. 3 and table 2), including locations on all major tributaries that could have an effect on flows in the Cooper River. Although an attempt was made to measure flow over a complete tidal cycle, most measurements covered a 10-hour period. Maximum positive or negative flows were measured in most cases. Measurements were made on July 24 and November 27, 1990, on Durham Canal and in Bushy Park Reservoir. Measurements were made on July 25, 1990, and April 25, 1991, on the Cooper River and its tributaries.



Base from U.S. Geological Survey 1:100,000 Digital Line Graphs, Saint George, Georgetown, Walterboro and James Island

Figure 2.--Gaging station locations.

Table 1.--Gaging stations and datums used in the study

[dashes indicate no data]

Station number	Datum of gage used (feet)	Datum by levels (feet)	Station location
[1]02172001	---	-5.00	Lake Moultrie Tailrace at Pinopolis Dam
[5]021720011	-11.59	-11.68	Lake Moultrie Tailrace, 0.2 mile downstream from Pinopolis Dam
02172002	-5.89	-5.91	Lake Moultrie Tailrace, 2.2 miles downstream from Pinopolis Dam
[2]021720025	---	-10.10	Lake Moultrie Tailrace, 2.2 miles downstream from Pinopolis Dam
02172003	-21.99	-21.99	West Branch Cooper River, 3.8 miles downstream from Pinopolis Dam
02172019	-18.59	-18.50	West Branch Cooper River, 11.3 miles downstream from Pinopolis Dam
02172020	-10.14	-10.14	West Branch Cooper River, 12.6 miles downstream from Pinopolis Dam
[4]02172025	-19.55	-19.36	West Branch Cooper River at Durham Canal
[2]02172026	---	-14.72	Durham Canal, 0.4 mile from West Branch Cooper River
[5]02172037	-21.60	-21.30	East Branch Cooper River, 2.1 miles upstream of the Cooper River
02172040	-14.05	-14.05	Durham Canal, 1.7 miles from West Branch Cooper River
02172050	-14.34	-14.34	Cooper River, 2.5 miles downstream from Durham Canal
[2]02172053	-13.57	-6.38	Cooper River, 3.0 miles upstream from Back River
[3]02172060	---	-4.58	Durham Canal, 2.2 miles from West Branch Cooper River

Table 1.--Gaging stations and datums used in the study--Continued

[dashes indicate no data]

Station number	Datum of gage used (feet)	Datum by levels (feet)	Station location
[2]021720601	-13.64	-13.87	Durham Canal 0.8 mile from Back River
[4]021720603	-4.68	-4.86	Upper Back River 2.2 miles upstream from Durham Canal
[4]021720612	-2.33	-2.37	Foster Creek 5.5 miles upstream from the Back River
02172062	-1.07	-1.07	Back River downstream from Foster Creek (above dam) near North Charleston
021720622	-16.20	-16.20	Back River downstream from Foster Creek (below dam) near North Charleston
[4]02172064	-5.31	-4.81	Flag Creek 5.4 miles upstream from the Cooper River
[5]02172065	-8.01	-8.01	Cooper River above Army Depot near North Charleston
[1]021720710	---	-17.12	Cooper River (auxiliary) at Custom House at Charleston
[1]021720711	---	-17.12	Cooper River at the Custom House at Charleston

[1]Station used only for quality control of stage data and is outside of the study area, therefore, not shown in figure 2.

[2]Station used only for quality control of stage data.

[3]Nonrecording station used for measuring site 9 only.

[4]Station used as external stage boundary for initial fit of the model.

[5]Station used as external stage boundary in the initial fit and final model.

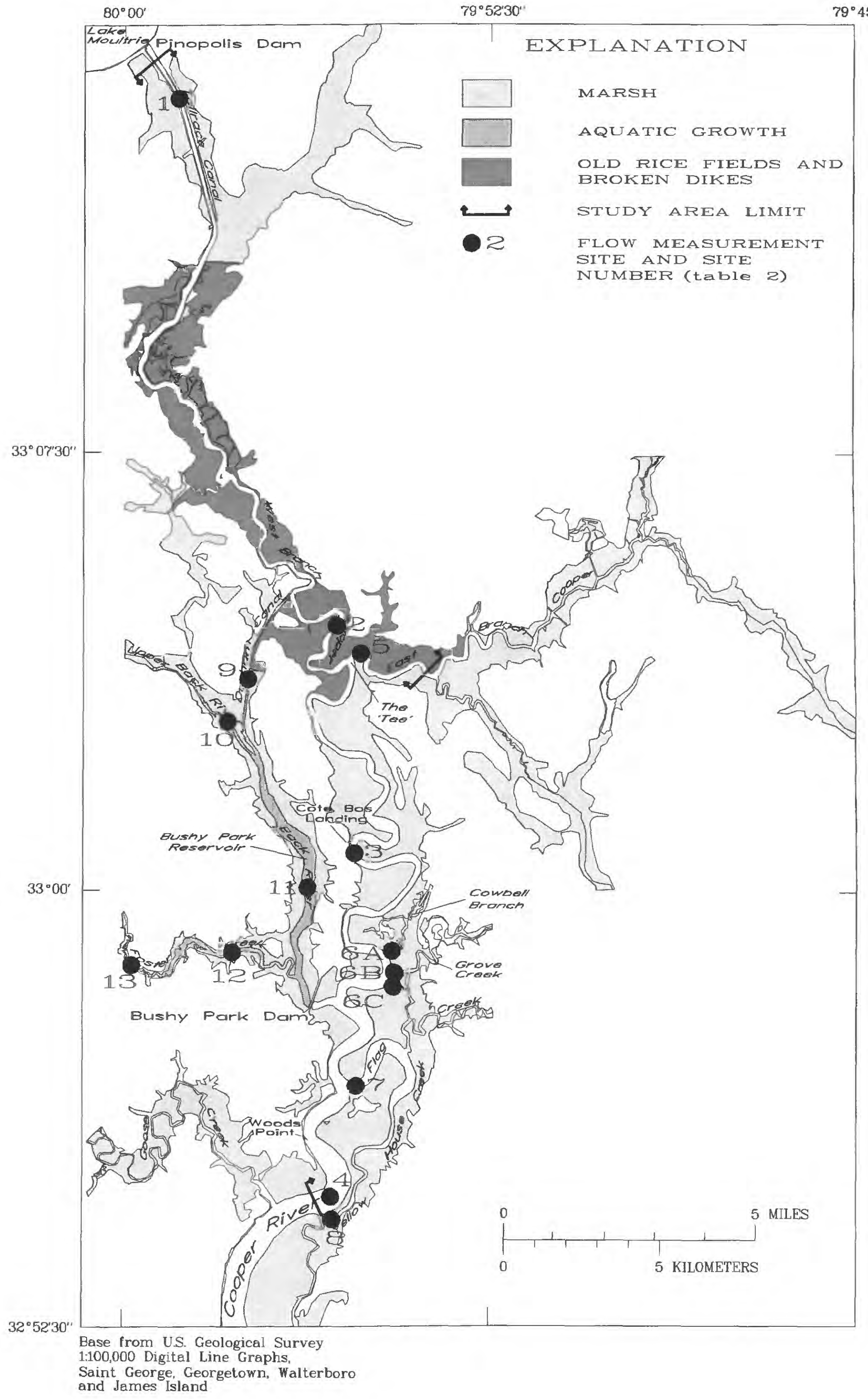


Figure 3.--Flow measurement site locations.

Table 2.--Flow measurement type, quality, and locations

Site identification (fig. 3)	Type of measurement	Quality of measurement	Location
1	B	F	West Branch Cooper River at station 02172002
2	L	P	West Branch Cooper River 2 miles upstream of the "Tee"
3	[1]L,C	[1]P,F	Cooper River at Cote Bas Landing downstream of site W5 discharge
4	L	P	Cooper River at Woods Point
5	L	P	East Branch Cooper River 0.3 mile upstream of the Cooper River
6	L	P	Sum of measurements 6B, and C
6A	L	P	Cowbell Branch at the mouth
6B	L	P	Grove Creek at the new mouth
6C	L	P	Grove Creek at the old mouth
7	L	P	Flag Creek 0.5 mile above the mouth
8	L	P	Yellow House Creek 0.1 mile above the mouth
9	B	F	Durham Canal at station 02172060
10	C	F	Upper Back River 0.8 mile upstream from Durham Canal
11	L	P	Back River between withdrawal sites W3 and W4
12	C	F	Foster Creek, 2.3 miles above the mouth
13	B	P	Foster Creek at station 021720612

B, Measurement made from a bridge.

C, Measurement made from a boat traversing a horizontally suspended cable.

L, Measurement made in a boat using the limited section method.

F, Fair (within 8 percent of correct flow).

P, Poor (measurement error could be greater than 8 percent of the actual flow).

[1]Measurement 1 was made by using the limited section method and was rated poor. Measurement 2 was made from a boat by using a horizontally suspended cable and was rated fair.

At bridge sites, or where a cable could be stretched across the channel, multiple passes were made across the river to obtain depths and velocities at fixed locations across the section. These depth and velocity readings were then interpolated at a uniform time interval (15 minutes) and flows were computed for each time interval at each fixed location across the section. The total flow for each measurement was then computed by summing all of the flows at the fixed locations for each time interval. Measurements made at the upstream end of Foster Creek (station 021720612) and the upper end of Upper Back River (station 021720603) are rated poor because of extremely low velocities.

A modification of the "limited section method" (Fulford and Sauer, 1986; and Bohman and Carswell, Jr., 1986) was used where a cable could not be stretched because the cross section was extremely wide, because the banks were too soft to anchor a horizontally suspended cable, or because boat traffic was too heavy. In this method, stream-bed elevations are determined from fathometer traces; water-surface elevations are recorded; and velocities are measured at 3 to 6 locations in the section. The area, which is computed from streambed and water-surface elevations, and linearly interpolated velocities are used to compute the final flow. The method was tested using tidal-cycle measurements made on Durham Canal where over 20 stations in the cross section were used to compute the baseline flow for comparison with the ratio interpolation method. In the documented ratio interpolation method, velocities are interpolated by the square root of depth ratios, but the Durham Canal tests showed that linear interpolation gives similar results. The tests showed that an accuracy of 2 to 7 percent could be obtained by linear interpolation of velocities between measured velocities at points in the measurement cross section. Measurements in the study area by this method are probably not as accurate as those in the test because of less uniform velocities in the horizontal. Radial swing of the boats about the anchorages, caused by tidal changes of flow direction, resulted in variations in depths, which also decreased the accuracy of flow determined by the method.

Cross-Section Geometry

Cross sections were obtained at approximately 1-mi intervals from graphic fathometer traces and from other studies. Where applicable, the cross sections were obtained by traveling at a constant rate of speed across the stream in a fathometer-equipped boat. Several crossings were made to insure accuracy. The distance across the stream was then determined by tagline, stadia, optical range finder, or scaling from a topographic map. Datums for the fathometer traces were obtained from water-surface elevations at nearby gaging stations at the time the cross sections were measured. Flood-plain elevations were obtained at selected sites by levels to benchmarks.

Flood-plain elevations at some gaging stations also were obtained by inspection of plots of minimum daily tide stages against maximum daily tide stages (fig. 4). The range between minimum and maximum tide stages is fairly constant when both minimum and maximum gage heights are either in or out of the main channel, as shown by the 1:1 slope of segments A and C of figure 4. When the maximum tide stage is out of banks, the range decreases, and the slope of segment B in figure 4 is less than 1. Segments A to C were graphically fitted to the data points. The maximum tide overtops the banks at the junction of segments A and B in figure 4 at a stage of 2.2 ft, and the minimum tide overtops the banks at the junction of segments B and C at a stage of 2.2 ft. The two stages agree with each other and support the method. The method is a more economically effective way of determining where water leaves the main channel than ordinary surveying methods and meets the accuracy requirements for this application. The method, however, is valid only when the range of tide stages is raised and lowered by considerable freshwater inflow or varying reservoir levels.

Some cross sections were linearly interpolated or duplicated from other cross sections. Widths of wide flood-plain or marsh areas were determined from topographic maps and hydraulic response characteristics. Cross sections were located at variations in channel shape, at gage locations, and at points where water is either injected or withdrawn.

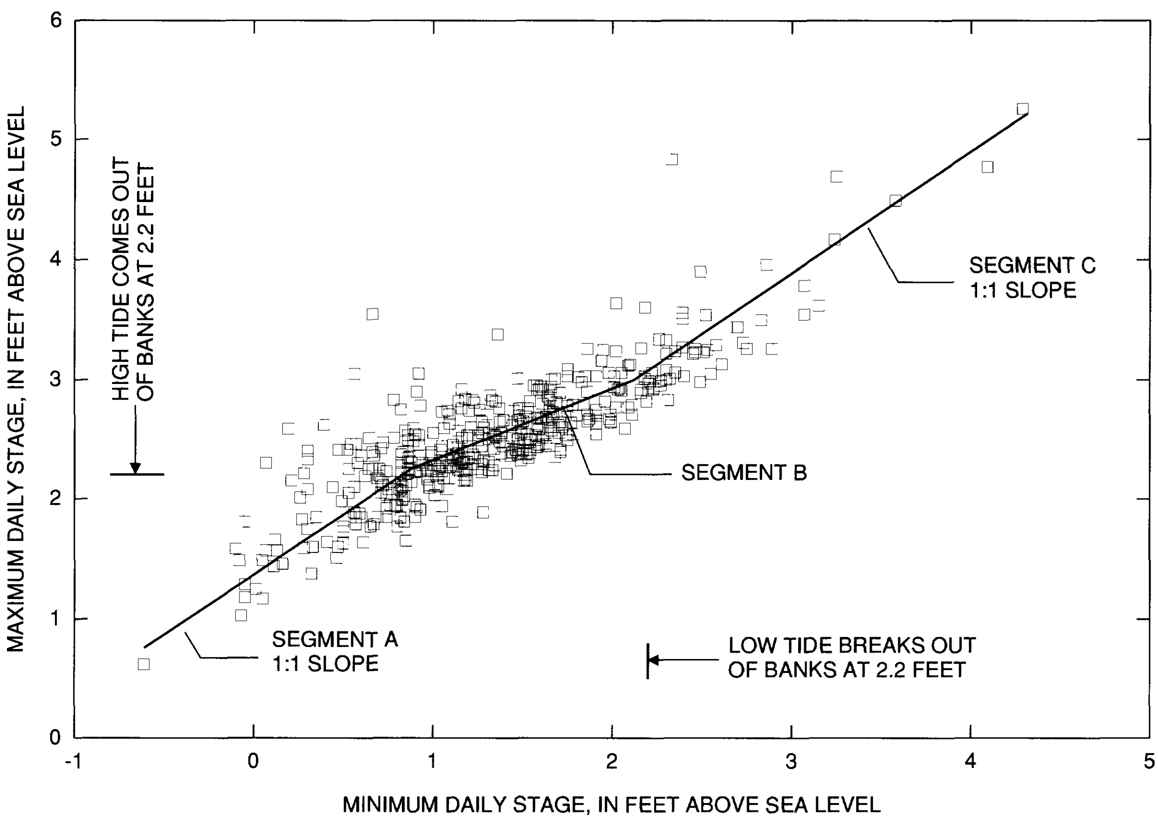


Figure 4.--Relation of high-tide stages to low-tide stages at station 021720612.

BRANCH MODEL

The U.S. Geological Survey BRANCH model, used to calculate flow in the study area, is a one-dimensional, unsteady-flow computer model for simulation of flow in interconnected channels (Schaffranek and others, 1981). The model can also be used to track the travel of particles injected at user-specified points. It was modified to route flow through flap-type tide gates.

The BRANCH model solves the one-dimensional equations of continuity and motion:

$$B \frac{\partial Z}{\partial t} + \frac{\partial Q}{\partial X} - q = 0 \tag{1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial(\beta Q^2/A)}{\partial X} + gA \frac{\partial Z}{\partial X} + \frac{gk}{AR^{4/3}} Q \mid Q \mid$$
$$-qu' - \xi B_c U_a^2 \cos \alpha = 0, \tag{2}$$

where

- B is the total channel top width, in feet;
- B_c is the top width of the conveyance part of the cross section, in feet;
- Z is the stage, in feet;
- t is the time, in seconds;
- Q is the discharge, in cubic feet per second;
- X is the longitudinal distance along the channel, in feet;
- q is the lateral side-channel flow, in cubic feet per second, per foot;
- A is the cross-sectional area, in square feet;
- g is the gravitational acceleration, in feet per second per second;
- k is a function defining flow-resistance;
- R is the hydraulic radius, in feet;
- U_a is the wind velocity in feet per second, occurring at an angle α from the positive x-axis;
- u' is the x-component of the lateral side-channel flow velocity in feet per second;
- β is the dimensionless momentum coefficient, and
- ξ is the dimensionless wind resistance coefficient.

The flow-resistance function is expressed as $k = (eta/1.486)^2$ where eta, is a flow-resistance coefficient.

In the derivation of equations (1) and (2), it is assumed that the flow is essentially homogeneous in density and that hydrostatic pressure is present everywhere in the channel. The channel is assumed to be reasonably straight, of simple geometry such as having a rectangular or trapezoidal shape, and to have a mild and uniform gradient.

Approximate solutions can be obtained for the nonlinear partial-differential unsteady-flow equations by finite-difference techniques. A weighted four-point finite-difference approximation is used in the BRANCH model. The finite-difference technique is described in detail by Schaffranek and others (1981).

The model uses values simulated at the current time level as initial conditions for computing the next time-step quantities, and proceeds step by step to the designated end of the simulation. Initial values of stage and discharge are required to start the simulation. These values can be obtained by measurement, computed from another source, derived from a previous unsteady-flow simulation, or estimated.

An idealized BRANCH model schematization is shown in figure 5. All cross sections adequately define conveyance, area, width, and storage capacity and are referenced to a common datum. A segment is a flow reach bounded by two cross sections. A branch is a single flow reach composed of multiple segments. An internal junction point is a point where two or more branches are joined. Flow may be extracted or added to the model at internal junction points. External junction points are ends of branches that do not connect to other branches. The model is driven by stages or flows input at external junction points. All other stages and flows are computed within the model.

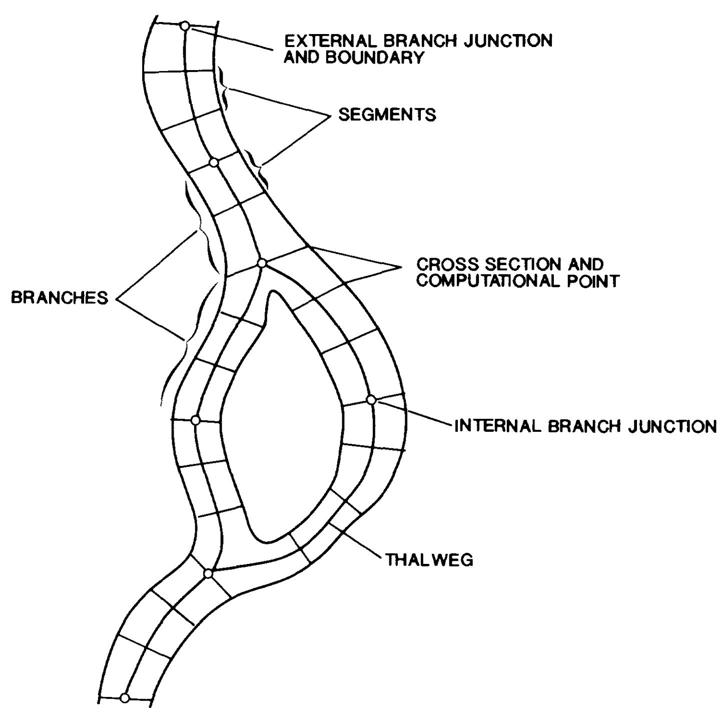


Figure 5.--Idealized branch network.

Water flowing overbank into marsh areas was assumed to be held in "dead storage" with no upstream or downstream velocity. In salt-marsh areas with fairly shallow depths and high resistance to flow by marsh grass, water is primarily distributed laterally by feeder tributaries at high tide rather than flowing strongly inland or seaward across the marsh-grass areas. In addition, velocity decreases to zero at high slack after the water gets out into the salt marshes. Use of "dead storage" was considered to be a more viable modeling method than weighting the roughness coefficients of the main channel and grassed marsh areas.

In the particle-tracking option of the program, simulated particles are injected to the model flow at user-defined points, and subsequent locations of the particles along user-defined tracks are computed based on mean velocities and elapsed time. No adjustments are made for constituent decay, dispersion, or diffusion in the particle-tracking method, and only flow calibration is necessary. Results are presented in tabular and graphic form.

Computation of Flow Through Tide Gates

Subroutines were written to compute flow through flap-type tide gates within the BRANCH flow model (Sanders, C.L., 1992, U.S. Geological Survey, written commun.). Flap gates are attached to the end of box or pipe culverts by hinges at the top, so that flow is allowed downstream, but not upstream. These gates do not require powerful hoist mechanisms, or human or electronic intervention to trigger their operation. Goodwin (1991) added flap-gate algorithms to the BRANCH model, but without flow-through culverts. Swain (1992) added full-pipe-flow algorithms to the BRANCH model, but without flow-through flap gates.

Such culverts may be partly or completely full of water, and the tide-gate subroutines will handle either case. For the partly full-pipe condition, flow can be quantified by three-parameter relations of headwater elevation, tailwater elevation, and flow determined by various hydraulic methods, such as described by Bodhaine (1968).

Full-pipe computations are much simpler (modified from Bodhaine, 1968) as shown by the following equation:

$$Q = K\ (H-T)^{0.5} \tag{3}$$

where

Q is flow, in cubic feet per second,
K is a constant for a given culvert geometry and material,
H is the headwater elevation, in feet, and
T is the tailwater elevation, in feet.

The constant "K" can be computed as a function of entrance-loss coefficient, friction-loss coefficient, culvert area, hydraulic radius, and length of culvert as described by Bodhaine (1968). The constant can also be adjusted for losses through the tide gates. It is also possible to simulate varying numbers of tide gates being open by an option in the subroutine to vary the "K" value with date and time.

Simulations were done only for full-pipe conditions because of the computational simplicity, and because other construction scenarios were not proposed at the time of this study. It is assumed that full-pipe is a viable alternative, because flow capacity is maximized, and because floating aquatic growth would not be as likely to clog the flap gate as it would for partly full flow. For simulations, tide gates are quantified in terms of the "K" factor, rather than various pipe shapes and configurations. A "K" value necessary to produce the desired flushing rate can be selected, and then converted into a corresponding number of box culverts or pipes of various sizes.

Calibration and Verification of the Model

The model was calibrated by using 15 flow measurements and stage data collected at 17 stations July 24-25, 1990, and verified using data collected November 7, 1990, and April 25, 1991. In general, the model was calibrated by fitting simulated data to measured data by adjustments in datum corrections, channel roughness coefficients, and storage based on various hydraulic considerations. It is assumed that if the model is reasonably well calibrated for both stage and flow, the calibration was accomplished by realistic adjustments to the hydraulically appropriate parameters. In a tidal environment, small flows cannot be measured or modeled as accurately as large flows. Therefore, more weight was given to the large positive or negative flows in the calibration process.

The model was first calibrated in three separate sections (fig. 6): the Tailrace Canal and West Branch Cooper River upstream of Durham Canal, the Cooper River and tributaries downstream of the canal, and the Bushy Park Reservoir, including Durham Canal. This was done to prevent errors generated in one section from being carried over to another section. As previously discussed, the model was calibrated at first using stage data from gages 021720603 on the Upper Back River, 021720612 on Foster Creek, 02172025 on Cooper River, and 02172064 on Flag Creek as external boundaries (figs. 2 and 6). The model was then extended past three of these gages to simulate storage upstream of the gages, and the gages were removed from the model as external boundaries (they continued to be used for internal stage comparison). By assuming zero inflow as the external boundary for these reaches, the model could then compute its own stages for scenarios when simulated tide gates were opened in the Bushy Park Dam without influence of stages for the closed-dam conditions of the calibrations phase. The assumption of zero flow is valid because freshwater inflows to the study area are usually negligible. The three sections were then combined with external boundaries at Pinopolis Dam (021720011), East Branch Cooper River (02172037), and Cooper River at Yellow House Creek (02172065), and the model was recalibrated in its final form (fig. 6).

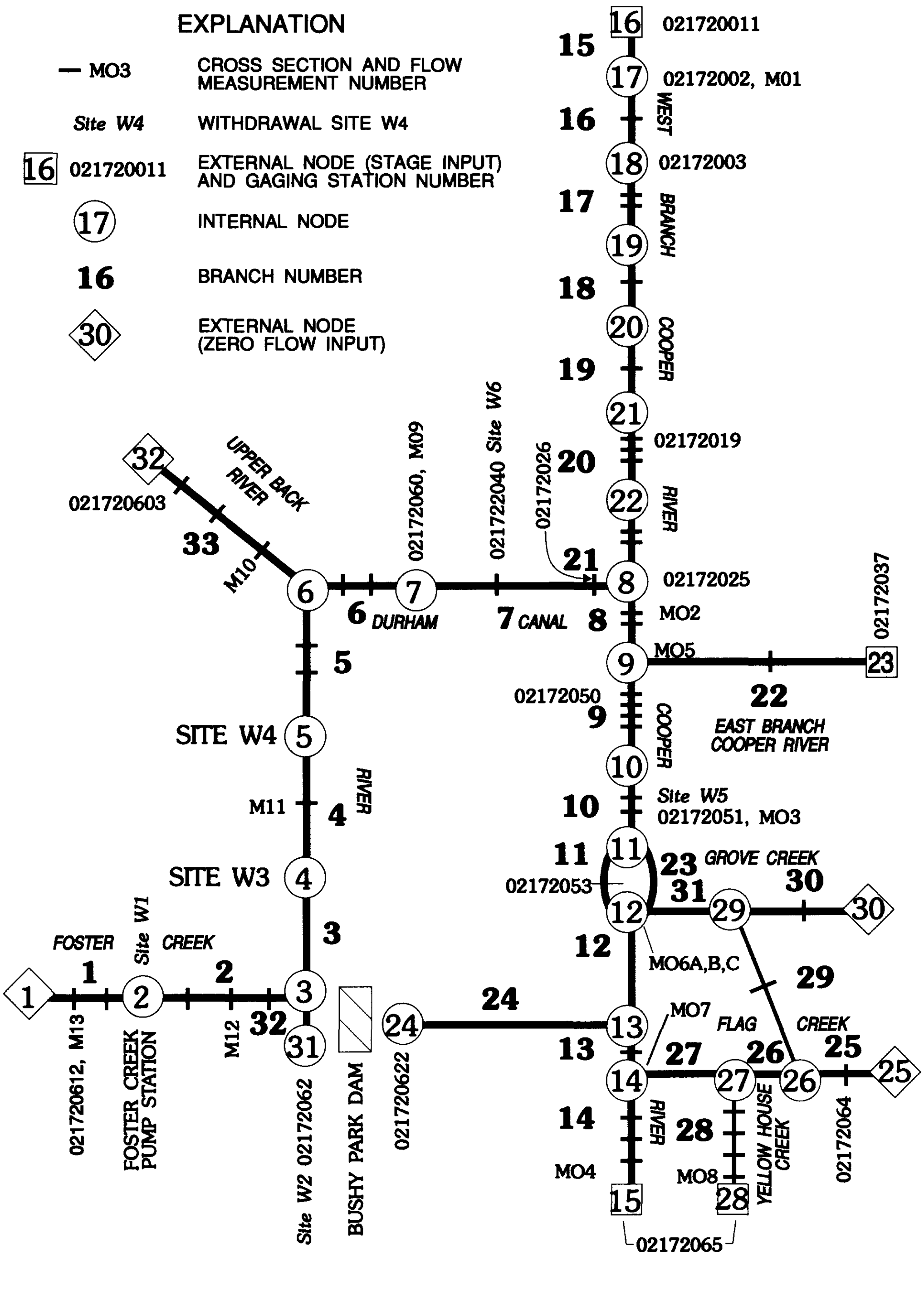


Figure 6.--Branches, junctions, and boundaries of the BRANCH flow model.

The fit of simulated to observed hydrographs of stage and flow for the measurements and stages for the calibrations of July 24, 1990, at selected sites, is shown in figures 7 and 8. The model could not be calibrated exactly, because of the uncertainties about the effects of the rice fields and broken dikes on the Cooper River, heavy aquatic growth in the Bushy Park Reservoir, poor flow-measurement conditions, numerous interconnected channels, and large areas of storage. Use of safety factors in the design could be used to compensate for the uncertainties in the model.

The calibration was verified by visual comparison of simulated and measured hydrographs of stage and flow for the measurements of November 7, 1990, and April 25, 1991, as shown by the examples in figures 9 and 10. The hydraulic parameters used to calibrate the model were not changed in the verification process.

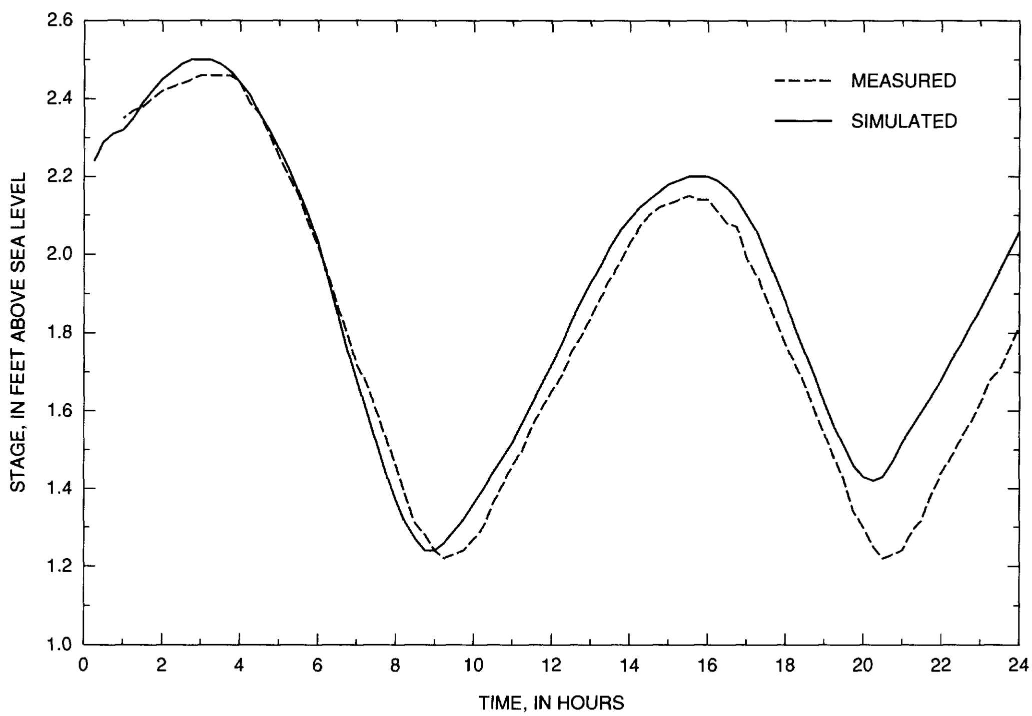


Figure 7.--Simulated and measured stages in Bushy Park Reservoir at station 02172062, July 24, 1990.

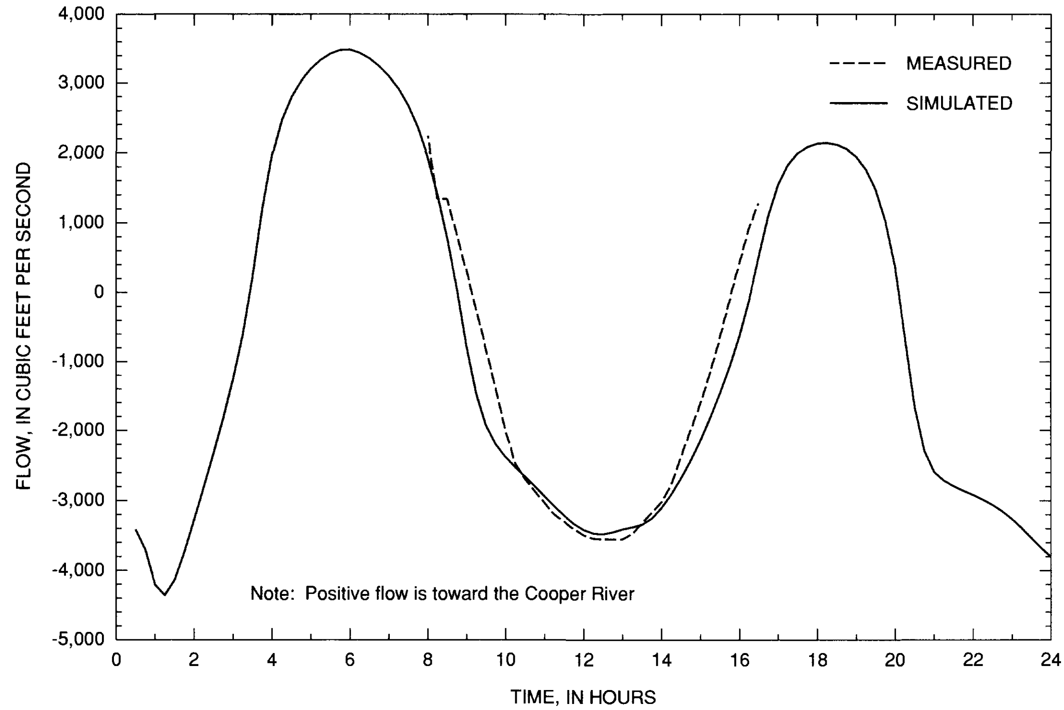


Figure 8.--Simulated and measured flows in Durham Canal at station 02172060, July 24, 1990.

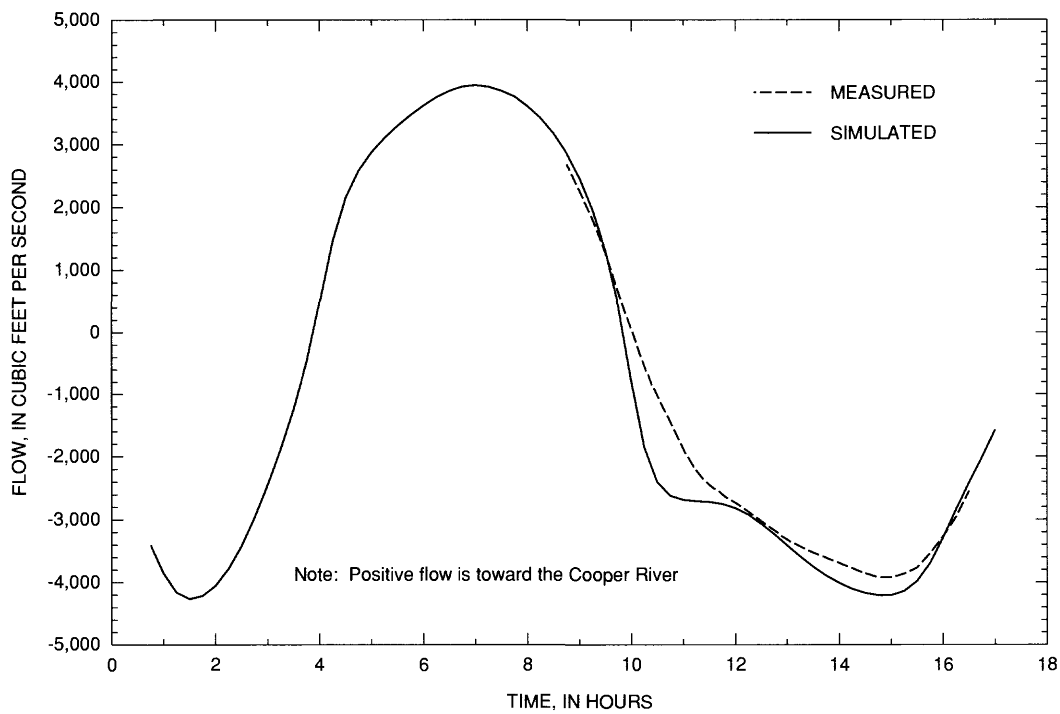


Figure 9.--Simulated and measured flows in Durham Canal at station 02172060, November 7, 1990.

Results of stage calibration and verification simulations are listed in table 3. Gaging stations are listed in table 1 and shown in figures 2 and 6. Results are quantified by the error of timing of the simulated hydrograph, the mean and sample-standard deviation of the residuals (predicted stages minus measured stages), and the difference in the ranges between high- and low-tide stages during the test period. Timing errors are caused by inaccurate flow and propagation rates through the model. The timing error was obtained by correlating measured stage with simulated stages lagged ahead or behind by several time periods. The time period having the highest correlation coefficient was assumed to be the timing error of the simulated hydrograph. The mean and sample-standard deviations of the residuals were computed after adjustment for the timing error. The mean of the residual is a measure of bias and the standard deviation of the residual is a measure of the scatter of the residuals. The difference in range is the difference between the range of simulated and measured stages over tidal cycles. This difference is a measure of how well the tidal range is simulated.

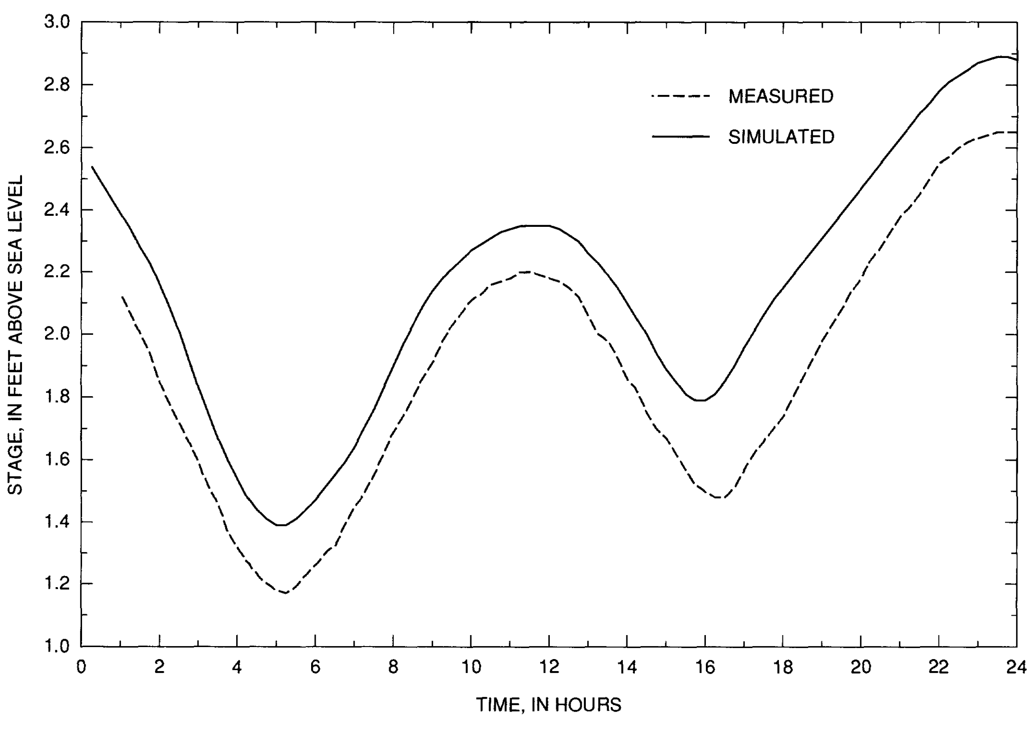


Figure 10.--Simulated and measured stages in Bushy Park Reservoir at station 02172062, April 25, 1991.

Time errors for stage calibration on July 24 and 25, 1990, and November 7, 1990, were within 15 minutes, except at station 021720612 where stages were simulated 60 to 90 minutes earlier than measured stages (table 3). Accurate timing was not attained for Foster Creek because of the difficulty of estimating the effects of extreme density of aquatic growth in the channel or hydraulic parameters such as eta, and because of the small flow in the creek compared to the flow closure tolerance of the overall model. The timing errors for the verification of April 25, 1991, were 30 minutes or less except for Foster Creek as listed in table 3.

The mean residual (bias) of stages varied from +0.36 to -0.14 ft for calibration and from +0.25 to -0.09 ft for verification as listed in table 3. The maximum standard deviation of residuals was 0.26 ft for calibration and 0.22 ft for verification. The difference in range varied from +0.47 to -0.49 ft for calibration and from +0.45 to -0.53 ft for verification. Given all the uncertainties involved in modeling such a complex system and the accuracy required for reasonable planning of withdrawal, an accuracy of ±0.3 ft for the mean and standard deviations of residuals, ±0.6 ft for differences in tidal ranges, and ±30 minutes in timing is considered adequate. Therefore, the simulated and measured stages agree within acceptable limits (table 3).

A summary of statistics of the flow calibration and verification is presented in table 4. Flow-measurement sites are listed in table 2 and shown in figures 3 and 6. Results are quantified by the error of the timing of the simulated hydrographs, by measures of volume error, and of sample-standard deviation of residuals (predicted flow minus measured flow). Volume errors and sample-standard deviations were computed after adjustments for timing errors. Volumes were computed over periods for which there were measured flows. Results are evaluated in comparison with the mean absolute simulated flow and the quality of the measured flow. Timing errors were computed by the same methods that were used for stage calibrations and verifications. Bias and scatter of relations of flow to some other parameter are usually computed in terms of logarithms of the dependent and independent variables, with a final conversion to percentage error. Because of the impossibility of taking logarithms of negative flows, a measure of the percentage bias was computed by multiplying 100 times the sum of the residuals divided by the sum of the absolute values of the observed flows. A measure of the percentage scatter was computed by multiplying 100 times the standard deviation of the residuals divided by the mean observed flow.

Table 3.--Summary of stage calibration and verification simulations of the BRANCH model at selected locations

[station number refers to gaging stations identified in Table 1]

BRANCH model location from figure 6						
Branch number	Cross-section number	Station number	Timing error[1] (minutes)	Mean of residuals[2] (foot)	Standard deviation of residuals (foot)	Range difference[3] (foot)
		Calibration of July 24-25, 1990				
1	2	021720612	-60	0.04	0.12	0.21
6	4	02172040	0	-.02	.15	-.48
7	3	02172025	15	.03	.19	-.48
9	2	02172050	-15	-.14	.20	.47
15	2	021720012	0	.16	.16	-.07
16	3	02172003	15	.11	.16	-.27
20	3	02172019	15	.03	.11	-.32
21	3	02172020	0	-.01	.11	-.25
24	1	021720622	0	.36	.12	.12
25	2	02172064	0	.06	.19	.00
32	2	02172062	-15	.09	.07	-.01
33	2	021720603	15	-.13	.08	-.13
		Verification of November 7, 1990				
1	2	021720612	-90	0.08	0.07	0.27
6	4	02172040	0	.21	.19	-.68
7	3	02172025	0	.03	.17	-.49
9	2	02172050	-15	-.10	.22	.42
15	2	021720012	-15	.15	.15	-.36
16	3	02172003	0	.09	.26	-.30
20	3	02172019	0	.05	.10	-.21
21	3	02172020	0	.01	.10	-.14
24	1	021720622	15	.13	.11	.16
25	2	02172064	0	.03	.16	.15
32	2	02172062	-15	.09	.04	-.01
33	2	021720603	15	-.06	.03	-.03

Table 3.--Summary of stage calibration and verification simulations of the BRANCH model at selected locations--Continued

[station number refers to gaging station identified in Table 1]

BRANCH model location from figure 6						
Branch number	Cross-section number	Station number	Timing error[1] (minutes)	Mean of residuals[2] (foot)	Standard deviation of residuals (foot)	Range difference[3] (foot)
		Verification of April 25, 1991				
1	2	021720612	-15	0.17	0.08	0.01
6	4	02172040	15	.16	.19	-.53
7	3	02172025	0	.00	.18	-.40
9	2	02172050	-15	-.09	.18	.45
15	2	021720012	0	.21	.17	-.09
16	3	02172003	15	.13	.22	-.45
20	3	02172019	15	.07	.11	-.23
21	3	02172020	0	.06	.11	-.17
24	1	021720622	0	.12	.12	.23
25	2	02172064	0	.19	.15	.15
32	2	02172062	0	.25	.06	.02
33	2	021720603	30	.05	.07	.01

[1]Negative sign means the simulated hydrograph occurred earlier than the measured hydrograph.

[2]Residual is computed by subtracting the measured stage from the simulated stage.

[3]Range difference is computed by subtracting the range between high and low measured tide stages from the simulated range.

Table 4.--Summary of flow calibration and verification simulations of the BRANCH model at selected locations

[dashes indicate no data]

BRANCH model location from figure 6								
Site number	Branch number	Cross-section number	Quality of measure-ment	Date	Timing error[1] (minutes)	Volume error[2] (percent)	Standard deviation[3] (percent)	Mean of absolute measured flow[4] (cubic feet per second)
				Calibration				
13	1	2	poor	7-24-90	-60	-67.6	123	7.7
12	2	3	fair	7-24-90	-30	-14.0	58.5	309
11	4	2	poor	7-24-90	-15	-1.7	99.1	1,480
9	6	3	fair	7-24-90	0	-16.0	21.2	2,160
9	6	3	fair	7-25-90	0	-8.8	11.7	2,790
2	8	2	poor	7-25-90	0	-19.3	32.8	13,700
3	10	1	poor	7-25-90	0	1.5	12.8	36,900
4	14	4	poor	7-25-90	15	-2.5	7.6	61,400
1	16	1	fair	7-25-90	0	13.0	21.6	7,240
5	22	3	poor	7-25-90	30	35.7	23.8	11,600
6	23	2	poor	7-25-90	60	9.4	17.5	2,640
7	27	2	poor	7-25-90	15	-15.8	36.3	9,770
8	28	5	poor	7-25-90	-15	-15.3	20.7	4,970
6	30	2	poor	7-25-90	---	-40.8	105	2,710
10	33	4	fair	7-24-90	0	-36.9	53.1	84.6
				Verification				
13	1	2	poor	11-7-90	-60	-34.5	53.8	17.8
12	2	3	fair	11-7-90	-75	23.8	76.0	298
11	4	2	poor	11-7-90	-60	-5.4	30.5	1,290
9	6	3	fair	11-7-90	-30	7.5	20.7	2,680
2	8	2	poor	4-25-91	0	9.8	5.7	22,100
3	10	1	fair	4-25-91	0	-0.5	6.2	40,000
4	14	4	poor	4-25-91	-15	6.7	31.8	72,300
1	16	1	fair	4-25-91	0	-6.3	4.2	19,500
5	22	3	poor	4-25-91	-15	-1.0	20.3	12,000
6	23	2	poor	4-25-91	15	28.9	18.5	2,340

Table 4.--Summary of flow calibration and verification simulations of the BRANCH model at selected locations--Continued

[dashes indicate no data]

BRANCH model location from figure 6								
Site number	Branch number	Cross-section number	Quality of measure-ment	Date	Timing error[1] (minutes)	Volume error[2] (percent)	Standard deviation[3] (percent)	absolute measured flow[4] (cubic feet per second)
				Verification				
7	27	2	poor	4-25-91	15	-24.3	51.6	7,840
8	28	5	poor	4-25-91	0	-12.1	21.1	3,600
6	30	2	poor	4-25-91	---	87.8	59.2	1,980
10	33	4	fair	11-7-90	-60	20.3	17.9	178

[1]Negative sign means the simulated hydrograph occurred earlier than the measured hydrograph.

[2]Percentage volume is computed by subtracting the measured volume from the simulated volume, dividing by the absolute measured volume exchange, and multiplying by 100.

[3]Percentage standard deviation is computed by dividing the standard deviation of the flow residuals by the average absolute measured flows.

[4]Mean of absolute flow is the mean of the absolute values of the measured flows.

[5]Not determined.

The calibration timing errors were within 30 minutes, as presented in table 4, except for the flows at site 13 at the upstream end of Foster Creek (fig. 6), which were simulated one hour early. Flows at 4 sites in Bushy Park Reservoir were simulated up to one hour early for verification. Flows at the remaining sites were simulated within 15 minutes of measured flows for verification.

The key measurements on the Cooper River portion of the model were at sites 1 to 4. Volume errors for these measurements ranged from -19.3 to +13.0 percent for calibration and from -6.3 to +9.8 percent for verification. These percentages balanced fairly well about zero. The standard deviations of residuals range from 7.6 to 32.8 percent for these measurements for calibration and from 4.2 to 31.8 percent for verification. Measurements with a standard deviation smaller than 13 percent appeared to fit very well from visual inspections. Therefore, standard deviation of approximately 13 to 15 percent less was about as small a deviation as can be expected within the limitations of this model and model input. Large differences between simulated and observed flow could occur because of small differences in simulated stages. Also, slight errors in timing of the simulated stages could cause large differences between simulated and measured flows. The standard deviations varied from 17.5 to 36.3 percent for the remaining measurements for calibrations on the Cooper River, except for measurement at site 6, which had a standard deviation of 105 percent.

The measurement at site 6 is actually the sum of measurements at sites 6B and 6C. Interconnected channels around Grove Creek, Flag Creek, and the Cooper River are inaccurately characterized as one channel in the model, which causes the large percentage standard deviation. The simulated flow, however, was a small percentage of the flow of the Cooper River. The standard deviation for verification varied from 18.5 to 59.2 percent for measurements at sites 5 to 9. More accurate calibration for these measurements was not necessary, because flows were relatively small compared to flows in the Cooper River.

The channel resistance factor (eta) used throughout the study generally ranged from 0.015 to 0.035 with a few extremes of 0.013 to 0.080 in locations of rapid (unobstructed) flow and slow sluggish flow with heavy aquatic growth, respectively. When stage is used as an external boundary, flow may be improperly computed by the model, and simulated flows could pass through an impervious boundary such as Pinopolis Dam. To minimize such flow a high eta value (0.999) was used for the upstream flow at station 021720011. Other program variable and dimensions of arrays in BRANCH are listed in table 5.

The key flow measurements in the Bushy Park Reservoir portion of the model were at sites 9 on the Durham Canal and 11 on the Back River, because they were indicative of the total flow in and out of the reservoir. For calibration, the flow measurement at site 9 for July 24 and 25, 1990, had an average volume error of -12.4 percent, but the flow measurement at site 11 had an error of only -1.7 percent. For verification, the flow measurements at sites 9 and 11 had a volume error of +7.5 percent and -5.4 percent, respectively. Standard deviation was within 21.2 percent for the flow measurement at site 9 for calibration and 20.7 percent for verification. These standard deviations were reasonably close to expected results of good calibration as shown for the Cooper River, where standard deviation of residuals were less than 32.8 percent for key measurements. The standard

Table 5.--Program variables and identifiers that set the maximum dimension of arrays in the BRANCH flow model used in this study

[Branch-Network Dynamic Flow Model (version 92/04/15), A four-point, implicit, finite-difference scheme with linear matrix solution by gauss elemination using maximum pivot strategy with optional iteration; dashes indicate no data]

Program variable	Definition	Maximum available	Used	Location
MXBH	Branches in network	35	33	/BRANCH/
MXJN	Junctions in network	35	32	/BOUNDY/
MAXS	Cross sections in network	125	115	/ARBLOG/
MXPT	Data per cross section	20	19	/CETA/
MXMD	Measured data locations	15	5	/MDATAS/
MAXCZQ	Time steps per day	720	96	/ZQ/
MXWIND	Time-varying wind data	1,500	0	/WIND/

Program variable	Definition	Position	Format	Value range	Assigned value
NBND	Number of external boundaries	7-8	I2	1<N<=35	10
NSTEPS	Number of time steps	9-12	I4	---	193
NIT	Maximum iterations allowed	17-18	I2	---	5
IEXOPT	Extrapolation option	24	I1	0/1	1
TYPETA	Friction resistance type	25	I1	1<=N<=7	4
IDTM	Time step	30-33	I4	---	15
THETA	Theta weighting factor	34-36	F3.2	0<=N<=1	0.85
QQTOL	Discharge convergence	37-41	F5.1	---	200
ZZTOL	Stage convergence	42-46	F5.3	---	.02
CHI	Chi weighting factor	62-64	F3.2	0<=N<=1	.50
GLBETA	Global default beta coefficient	1-4	F4.2	N>=1	1.0

Table 5.--Program variables and identifiers that set the maximum dimension of arrays in the BRANCH flow model used in this study--Continued

Boundary-value data definition

Station number	Type[1]	Junction	NDATA[2]	DTT[3]	Datum	Constant value
21720011	Z	16	0	15	-11.590	---
21720603	Q	32	1	0	1.000	Q(t) = 0.000
2172062	Q	31	1	0	1.000	Q(t) = .000
21720612	Q	1	1	0	1.000	Q(t) = .000
2172037	Z	23	0	15	-21.600	
	Q	30	1	0	1.000	Q(t) = .000
21720622	Q	24	1	0	1.000	Q(t) = .000
2172064	Q	25	1	0	1.000	Q(t) = .000
2172065	Z	15	0	15	-8.010	---

Time-varying nodal-flow data definition

Station number	Type	Junction	NDATA	DTT	Multiplier
W1	Q	2	0	60	1.000
W2	Q	3	0	60	1.000
W3	Q	4	0	60	1.000
W4	Q	5	0	60	1.000
W6	Q	7	0	60	1.000
W5	Q	10	0	60	-1.000

Culvert nodal-flow defined at junction 24
Culvert nodal-flow defined at junction 31

[1] Z, stage; Q, discharge.

[2] Location of data base for model.

[3] Time interval of data, in minutes.

deviation for the flow measurement at site 11 was 99.1 percent for calibration and 30.5 percent for verification. The flow measurement at this site was rated poor, because the ratio method was used in very slow, less-than-uniform velocities. Volume errors for the flow measurements at sites 10, 12, and 13 in the Bushy Park Reservoir ranged from -67.6 to -14.0 percent for calibration and from -34.5 to +23.8 percent for verification. Standard deviation ranged from 0.1 to 122.7 percent for calibration and from 17.9 to 76.0 percent for verification. Although these percentages are sizeable, the mean flows ranged from only 7.7 to 309 ft^3/s. These percentages were to be expected, because the mean flows were on the order of the allowable error of computational closure for the model of ± 200 ft^3/s. (A computational closure of 200 ft^3/s is very small for the Cooper River, where flows of 20,000 to 100,000 ft^3/s are occasionally experienced.) Moreover, flow in Bushy Park Reservoir has a greater potential for the occurrence of two-dimensional flow in the horizontal plane, because the middle of each channel is unobstructed, while the edges of the channels are obstructed by thick aquatic growth.

Particle Tracking

In the particle-tracking option of the BRANCH model (R.W. Schaffranek, written commun., 1991), a hypothetical particle injected at a cross section is transported through the model according to simulated velocity and elapsed time. Calibration is not necessary for particle tracking, because particle tracking depends only on velocity of flow. Adjustments are not made for dispersion, diffusion, or decay and are not necessary for this study because of fairly rapid flushing rates to be induced by the tide gates in the area of interest. Simulated particles in the upper reaches of Foster Creek or Upper Back River never left the reach; they moved upstream and downstream as the reservoir level rose and fell. In reality, these particles would gradually move out of the reach due to dispersion, diffusion, and the net downstream flow derived from amounts of tributary and ground-water inflow.

Particles could be withdrawn from the model at external junction points. Therefore, if a large amount of water were withdrawn at an internal junction point, a particle would be attracted to that point and be held there. In this case, it should be assumed that the particle would have been withdrawn with the water at that point.

Five tracks were defined for particle tracking as shown in figure 11. Particles could travel in either direction, depending on flow in the model. Tracking was quantified by the number of miles traveled with time from the most upstream point of the established track. Only tracks number 1 and 3 were used in the final analysis, because these tracks included the Bushy Park Reservoir study area. An example of the distance traveled along track 1 in Bushy Park Reservoir by a particle released on September 15, 1990, at Durham Canal is shown in figure 12. The particle was transported down Back River and up Foster Creek to the Foster Creek intake by municipal withdrawals and operation of the tide gates at Bushy Park Dam. Because the particle cannot be removed at the internal node at the Foster Creek intake by the model, it moves back and forth with the tide in the vicinity of the intake, and is assumed to have left the system.

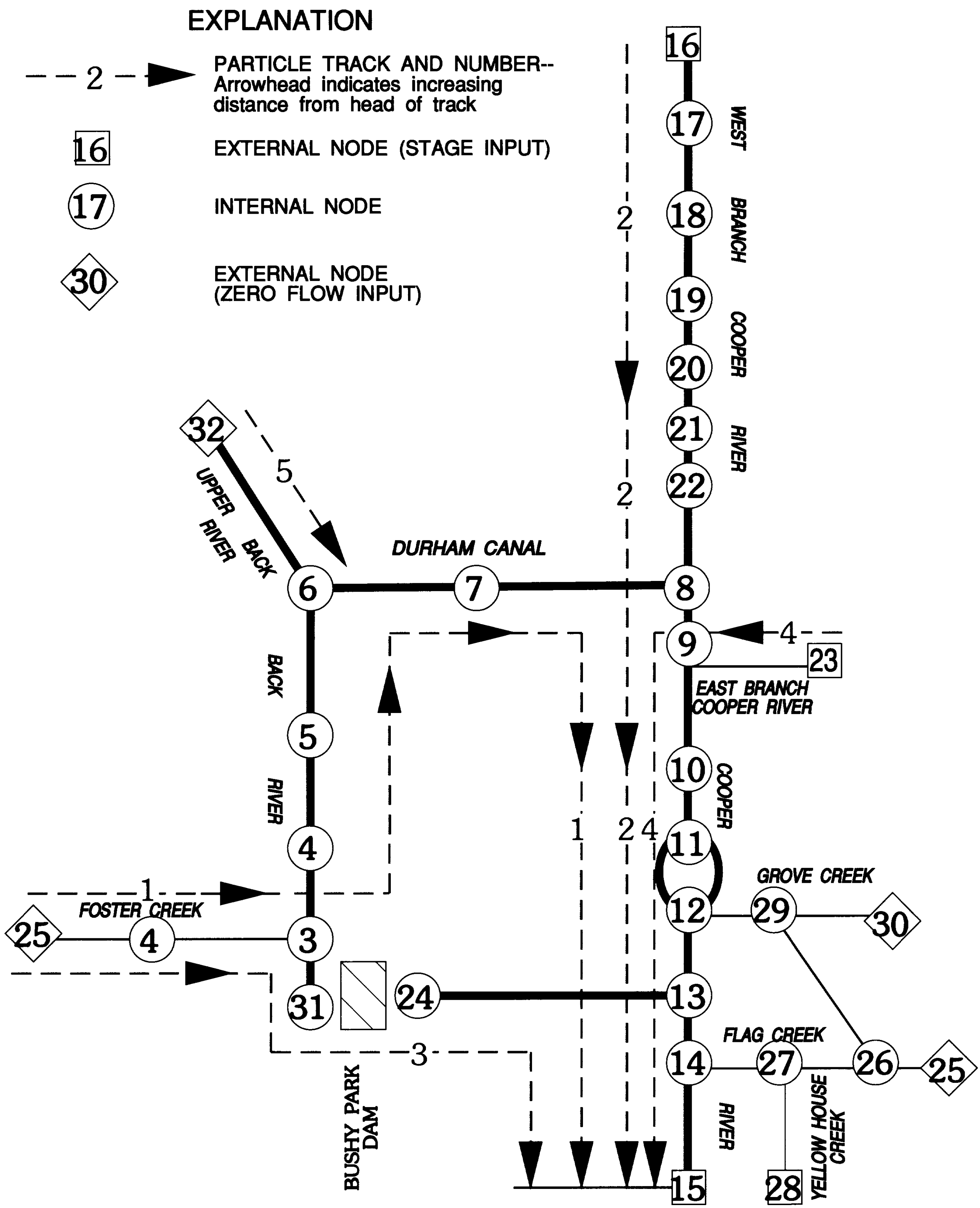


Figure 11.--Schematic representation of particle tracks for the study area.

Sensitivity Analysis

Simulated flows are sensitive to variations of eta values (a measure of resistance to flow, basically equivalent to Manning's n), gage datum corrections, and datums of cross sections (Schaffranek and others, 1981). However, sensitivity of flushing rates in Bushy Park Reservoir may not correspond to the sensitivity of simulated flows. For example, the reservoir could be described as a large basin with a small amount of water flowing in and out at Durham Canal, and even smaller amounts of water being removed at other points. The water in storage moves back and forth as the tide flows in and out, but travel time through the reservoir should be determined mostly by flow rates through the tide gates and various withdrawals, by the amount of storage in the reservoir, and the flow capacity of Durham Canal. Flushing rates could be changed by parameters that influence fall across Bushy Park Dam, but flow from the dam varies somewhat insensitively as the square root of fall (eq. 3). The sensitivity analysis was quantified by effects on flushing rates rather than flow rates simulated by the model, because in this report, the effectiveness of the various scenarios are determined in terms of the flushing rate rather than the flow rate.

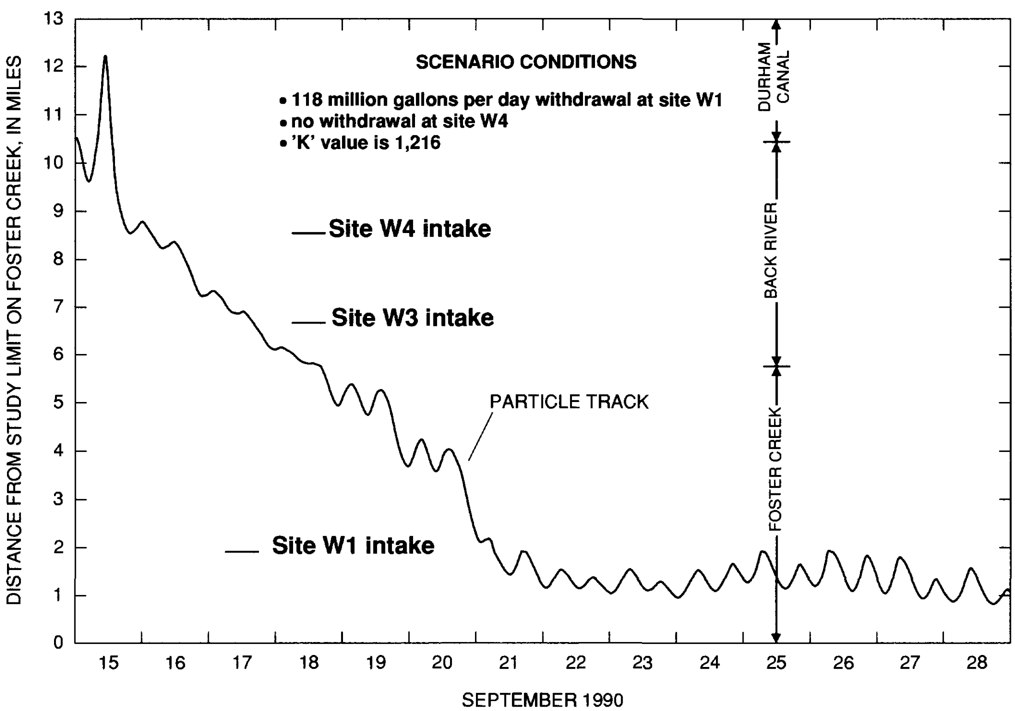


Figure 12.--Travel distance in Bushy Park Reservoir of a particle released at Durham Canal.

The analysis was conducted for the following scenario:

a) A "K" value (eq. 3) of 1,216 was used, because it gives a suitable flushing rate for most scenarios.

b) The Charleston CPW intake was assumed to be on the Back River, at withdrawal site W3 (fig. 1), away from effects of Foster Creek.

c) A withdrawal of 50 Mgal/d, the current load, was assumed, as a worst-case withdrawal rate.

d) Normal withdrawals at site W4 were assumed.

The eta value was varied by $\pm$25 percent for Bushy Park Reservoir, Durham Canal, and the main channel of the Cooper River from Pinopolis Dam to the lower boundary of the study area. Datums of cross sections in Bushy Park Reservoir and Durham Canal were changed by $\pm$1.0 ft to determine the effects of changes in both conveyance and storage. Datum corrections were also changed by $\pm$0.5 ft for the external boundary stage gages (stations 021720011, 02172037, and 02172065).

Changes in flushing times at withdrawal site W3 and at the mouth of Foster Creek resulting from the changes in the sensitivity analysis are presented in table 6. All results are within $\pm$8 percent, except for $\pm$16 percent for adding $\pm$1.0 ft to the datums of the cross sections of Bushy Park Reservoir and -15 percent when the datums of stations 02172065 and 02172037 were lowered by 0.5 ft as presented in table 5. Flushing times are somewhat more sensitive to these two datum variations, because they have a greater influence on the fall across the Bushy Park Dam and therefore, flow through the dam. In general, flushing rates are fairly insensitive to variations in eta values, cross-section datums, and gage datums.

Table 6.--Summary of sensitivity analysis of flushing rates to variations in eta[1] values, datums of cross sections, and gaging station datum corrections

	Change in flushing time[2]			
	Withdrawal at Site W3		Withdrawal at mouth of Foster Creek	
	Days	Percentage[3] difference	Days	Percentage[3] difference
25 percent larger eta value for Bushy Park Reservoir	0	0	0	0
25 percent smaller eta value for Bushy Park Reservoir	0	0	0	0
25 percent larger eta value for Durham Canal	+.1	+4	0	0
25 percent smaller eta value for Durham Canal	-.2	-8	-.4	-10
25 percent larger eta value for main channel Cooper River	0	0	0	0
+1.0 foot datum correction applied to Bushy Park cross sections	-.4	-16	-.5	-12
-1.0 foot datum correction applied to Bushy Park cross sections	+0.4	+16	+0.2	+5
+1.0 foot datum correction applied to Durham Canal cross sections	+.1	+4	0	0
-1.0 foot datum correction applied to Durham Canal cross sections	-.1	-4	-.1	-2
Datum at station 021720011 changed from -11.59 to -11.09 feet	-.2	-8	-.4	-10
Datum at station 021720011 changed from -11.59 to -12.09 feet	0	0	0	0

Table 6.--Summary of sensitivity analysis of flushing rates to variations in eta[1] values, datums of cross sections, and gaging station datum corrections--Continued

	Change in flushing time[2]			
	Withdrawal at Site W3		Withdrawal at mouth of Foster Creek	
	Days	Percentage[3] difference	Days	Percentage[3] difference
Datum at station 02172037 changed from -21.60 to -21.10 feet	-0.3	-12	-0.6	-15
Datum at station 02172037 changed from -21.60 to -22.10 feet	+.2	+8	+.1	+2
Datum at station 02172065 changed from -8.01 to -7.51 feet	+.2	+8	+.1	+2
Datum at station 02172065 changed from -8.01 to -8.51 feet	-.3	-12	-.6	-15

NOTES:

[1]Eta is a measure of resistance to flow corresponding to Manning's n.

[2]The original flushing rates from Durham Canal to Site W3 and the mouth of Foster Creek were 2.5 and 4.0 days, respectively, for a "K" value of 1,216 and a withdrawal rate of 50 Mgal/d. The change in flushing rates is the new flushing rate minus the old flushing rate.

[3]The percentage change in flushing rate is 100 multiplied by the ratio of the change in flushing rate to the original rate.

SIMULATION OF RETENTION TIME IN BUSHY PARK RESERVOIR

Many flow and particle-tracking simulations were made to determine the relative effectiveness of various scenarios designed to improve the flushing characteristics of Bushy Park Reservoir. Decreasing the retention time of water in the reservoir should cause the quality of water in the reservoir to be more like that of Durham Canal, which is of sufficient quality to be more economically treated than the current supply. The scenarios included:

1. Increasing Charleston CPW withdrawals from a daily mean flow of 10 to 50, 118, and 150 Mgal/d. Current withdrawals from the Foster Creek (withdrawal site W1) and Edisto River average about 50 Mgal/d. Short-term projected demand is 118 Mgal/d and ultimate demand is estimated to be 150 Mgal/d.

2. Moving Charleston CPW intake from site W1 to the location of the site W3 intake. The only significant flushing of Foster Creek is by withdrawals at the Foster Creek intake and by intermittent storm-water runoff from Foster Creek Basin. Base flow from the basin is negligible. The quality of water at site W3, upstream of possible contamination by Foster Creek water, could be effectively improved by outflows from proposed tide gates at Bushy Park Dam.

3. Allowing flow through Bushy Park Dam by various sizes and numbers of flap-type tide gates.

Scenarios also were simulated with and without withdrawals by the thermoelectric power plant at site W4 to determine the effects on flushing rates during periods of zero withdrawal.

Flows through Bushy Park Reservoir with the hypothetical tide gates operating would, of course, vary with fall across the Bushy Park Dam, which varies with tidal cycles and possibly with large variations of flow releases from Pinopolis Dam. Stages on the upstream and downstream side of Bushy Park Dam for the period September 20 to 26, 1991, are compared as shown in figure 13. Note that the tide cycles are out of phase with each other, and therefore, hypothetical flow through the Bushy Park Dam would be maximized. The time frame of the project was too short to provide enough data to determine accurate durations of flows through the Bushy Park Dam; therefore, daily mean flows through the dam were simulated as shown in figure 14 to determine the period of minimum sustained flow (Sept. 15-29). All simulations were done during this period to represent lowest flushing rates for the period of record caused by periods of lowest outflow from the Bushy Park Dam. It should be remembered, however, that a recurrence interval is not attached to this selected period, and periods of even lower outflows from the dam and longer flushing rates could be experienced. Flows simulated for the 1991 water year (Oct. 1990 - Sept. 1991) at Pinopolis Dam are shown in figure 15. The simulated monthly flow at Pinopolis Dam for September 1991 is 5,270 ft^3/s.

Results of simulations of flap-type tide gates with full-pipe flow for September 15-29, 1991, are shown in figures 16 to 19. The days-to-flush may be determined for any withdrawal from 50 to 150 Mgal/d for the intake at Foster Creek or the hypothetical intake at site W3, and any "K" factor from 0 to 1,820 at the dam with or without the site W4 thermoelectric power plant withdrawals. Days-to-flush is the number of days for a particle to travel from the junction of Durham Canal and Bushy Park Reservoir to either the present Foster Creek intake or a new intake at site W3. The "K" factor has been previously described.

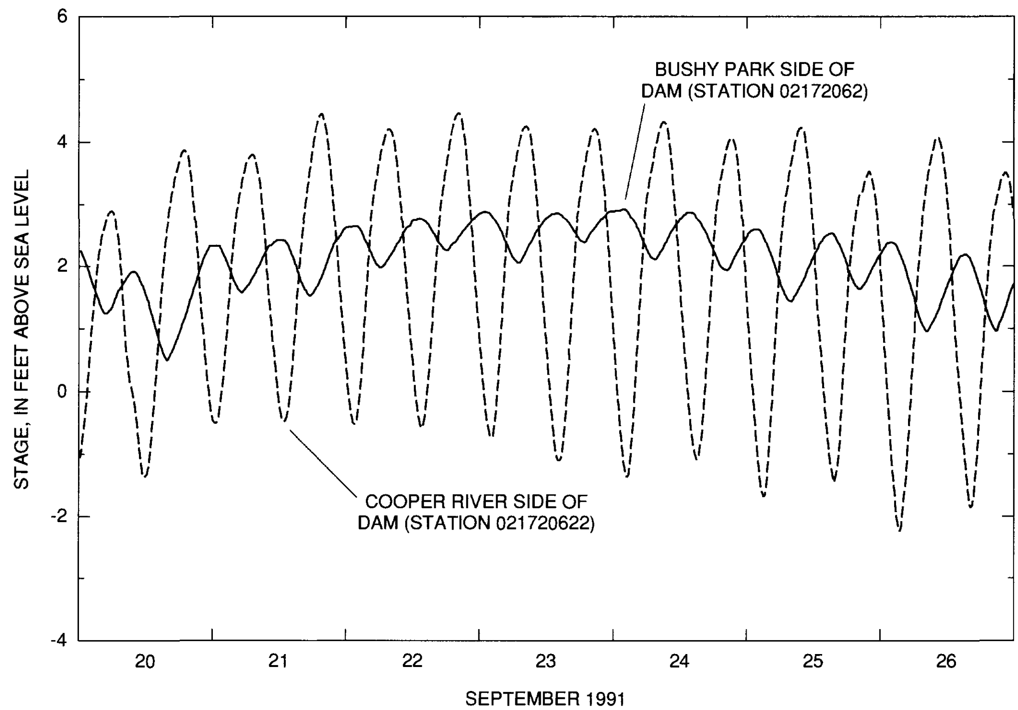


Figure 13.--Stages upstream and downstream of Bushy Park Dam, September 20-26, 1991.

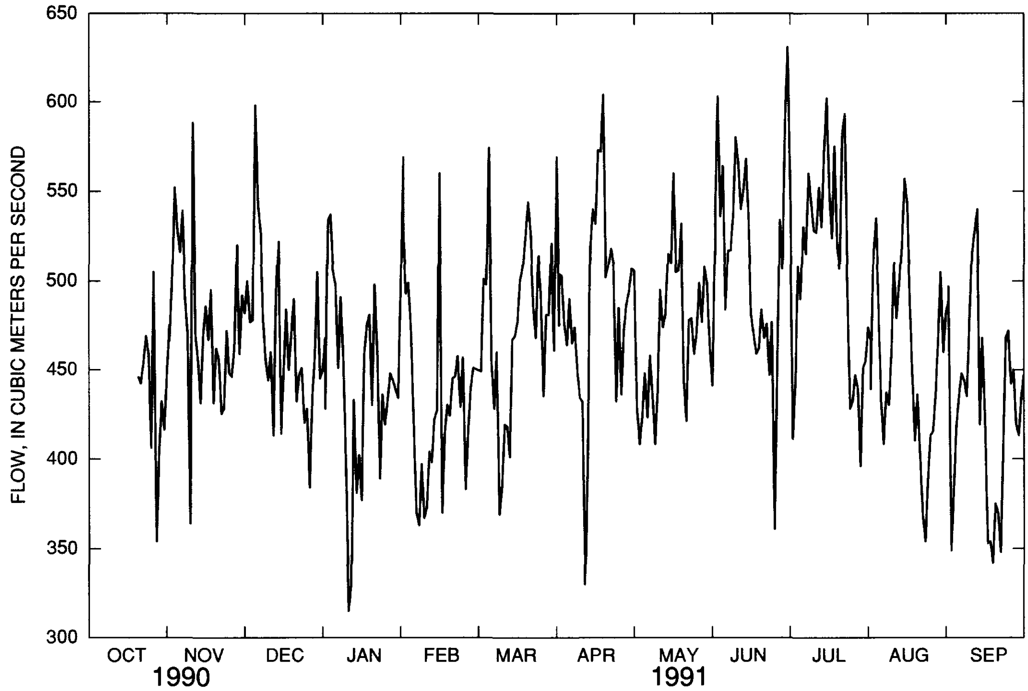


Figure 14.--Hydrograph of simulated mean daily flows through Bushy Park Dam for the 1991 water year.

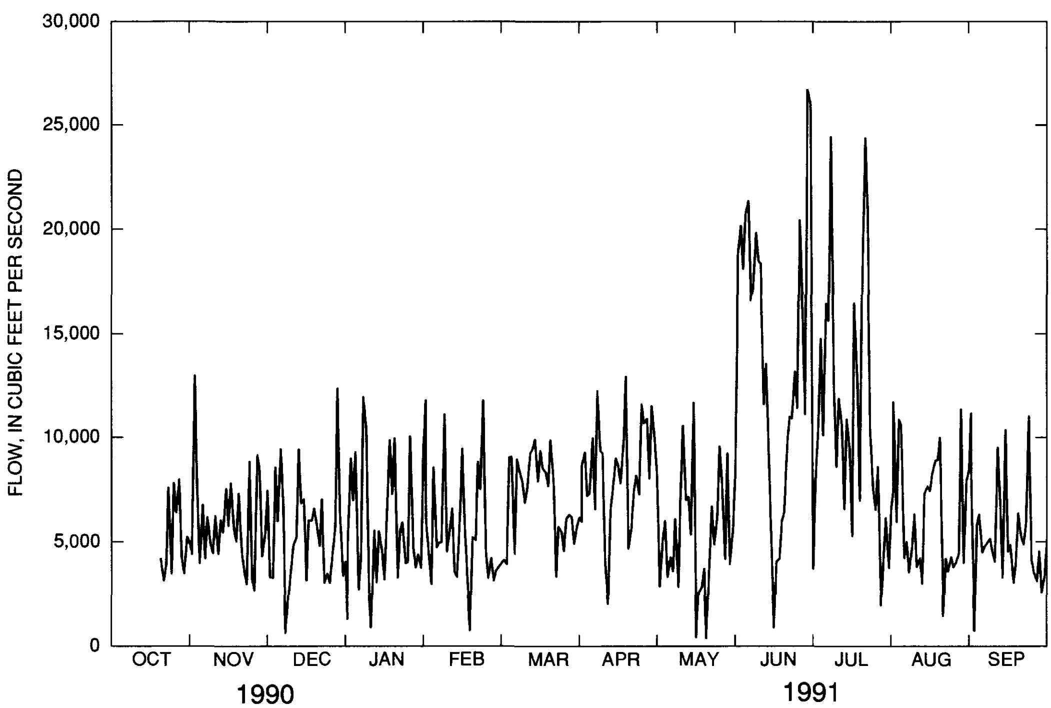


Figure 15.--Hydrograph of simulated daily mean flows at Pinopolis Dam for the 1991 water year.

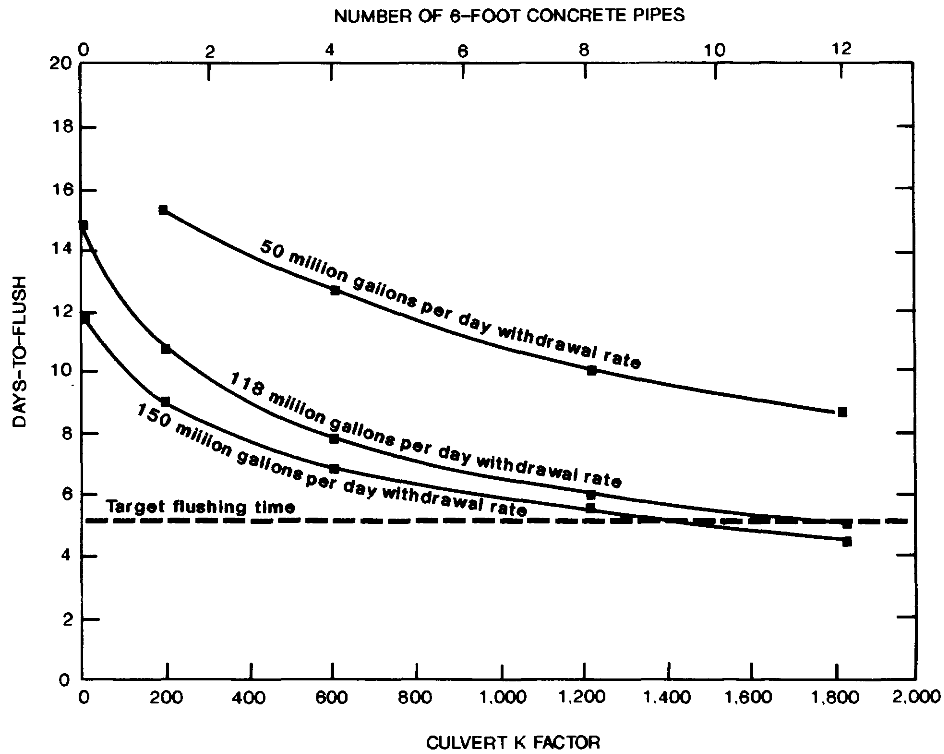


Figure 16.--Flushing times of Bushy Park Reservoir from Durham Canal to Foster Creek Intake for selected tide gate "K" factors and Charleston Commissioners of Public Works withdrawals with normal withdrawal at site W4.

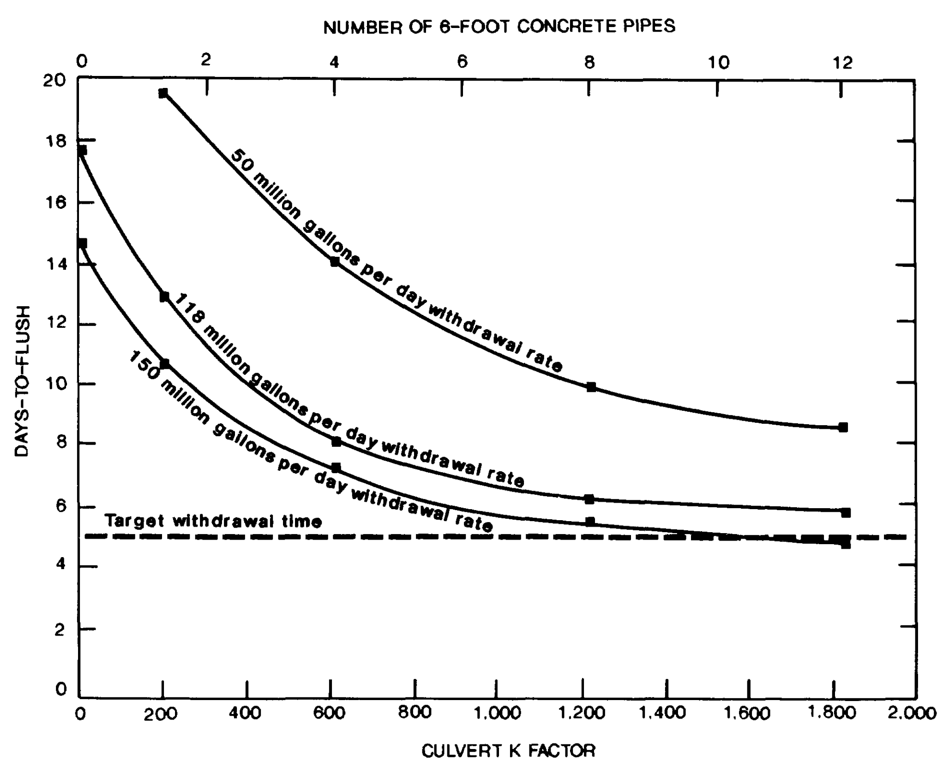


Figure 17.--Flushing times of Bushy Park Reservoir from Durham Canal to Foster Creek Intake for selected tide gate "K" factors and Charleston Commissioners of Public Works withdrawals with no withdrawal at site W4.

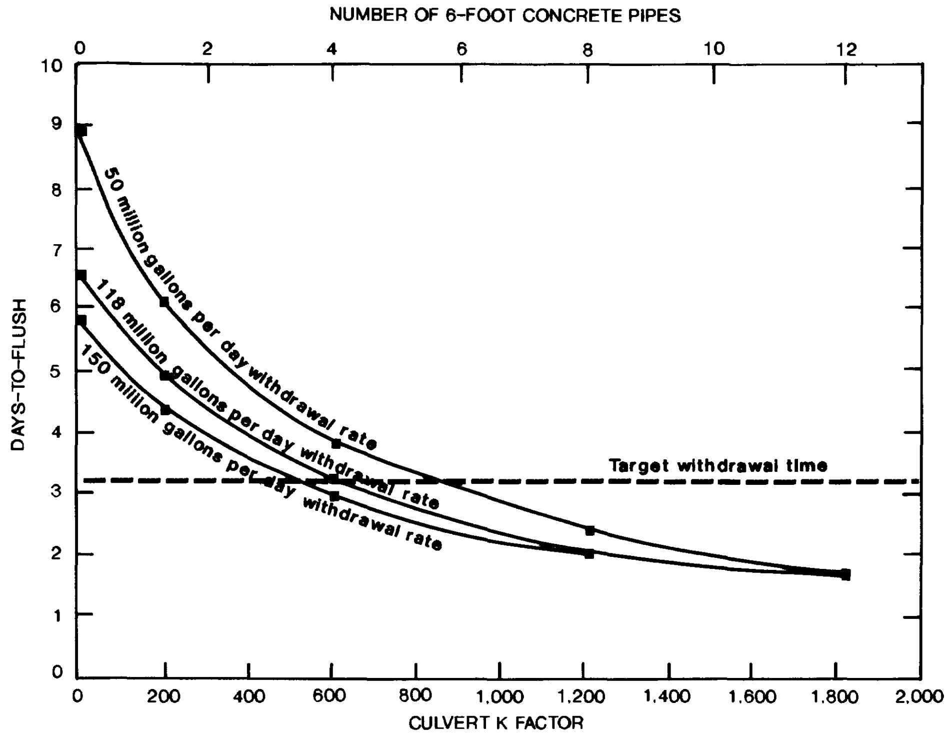


Figure 18.--Flushing times of Bushy Park Reservoir from Durham Canal to Site W3 intake for selected tide gate "K" factors and Charleston Commissioners of Public Works withdrawals with normal withdrawal at site W4.

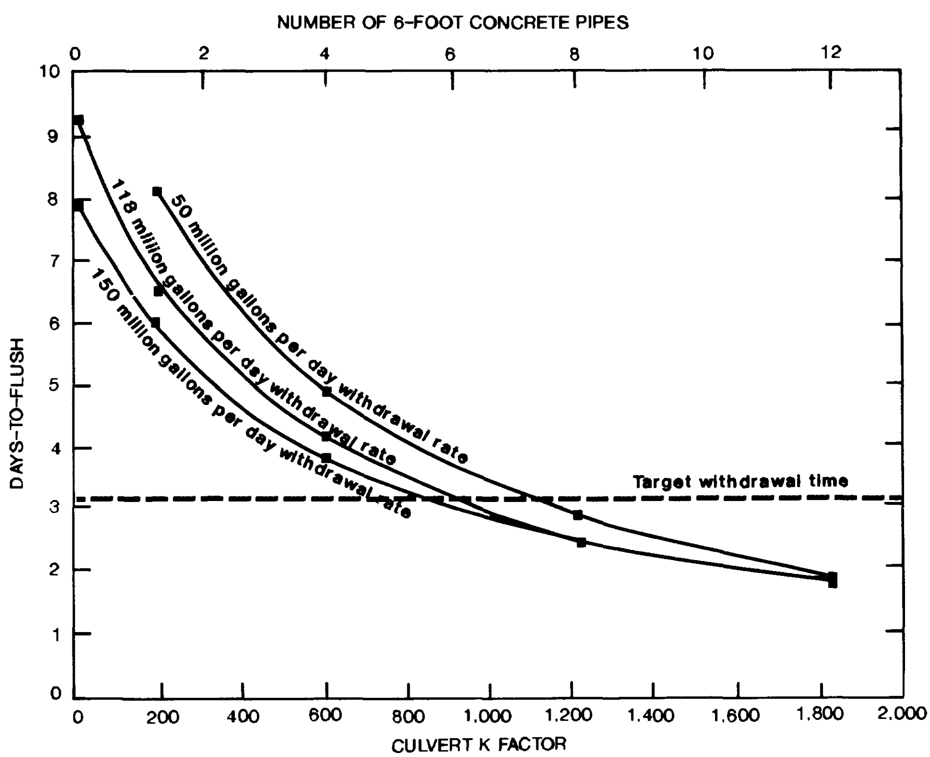


Figure 19.--Flushing times of Bushy Park Reservoir from Durham Canal to site W3 intake for selected tide gate "K" factors and Charleston Commissioners of Public Works withdrawals with no withdrawal at site W4.

The "K" factors of 0, 200, 608, 1,220, and 1,820 were used to construct figures 16 to 19. The "K" factor for full-pipe flow, without adjustment for losses due to the tide gate, can be computed by the following equation by Bodhaine (1968):

$$K = C\,Ao\sqrt{\frac{2g}{1 + \dfrac{29\,C^2\,n^2\,L}{R_o^{4/3}}}}, \tag{4}$$

Where
- C is the coefficient to quantify entrance losses,
- Ao is the cross-sectional area of the culvert, in square feet,
- g is the acceleration of gravity, in feet per second per second,
- n is the Manning's roughness coefficient (a measure of resistance to flow),
- L is the length of the culvert, in feet,
- R_o is the hydraulic radius of the culvert, in feet.

"K" factors for several culvert sizes are listed in table 7. For example, "K" factors of 608, 1,220, and 1,820 represent four, eight, and twelve 6-foot diameter concrete pipe culverts, respectively, without adjustment for flap-gate losses. A "K" factor can be determined from figures 16 to 19 to achieve a desired number of days-to-flush and then converted to an equivalent number of pipes or culverts from table 6. Partly full-pipe flow or other operational scenarios can be directly simulated by the BRANCH flow model, if necessary.

There seems to be a diminishing return for increase in "K" factor larger than about 600, as shown in figures 16 to 19, probably because increasing flows from the Bushy Park Dam lowers the head on the Bushy Park side of the dam to a point where the fall that drives the flow through the dam is diminished and fairly stabilized. Figures 16 to 19 also show that the days-to-flush are very sensitive to changes of the "K" factor when the "K" factor is less than 200. The days-to-flush are also very sensitive to the withdrawal rate for site W1. Generally, the site W4 withdrawals decrease days-to-flush by less than a day.

Table 7.--"K" factors for selected culvert types and sizes, without adjustment for losses through tide gates

Size[1] (feet)	Manning's n value	Entrance coefficient	"K" value
	Concrete pipe culverts		
4	0.015	0.88	59.0
6	.015	.87	151
8	.015	.86	285
10	.015	.86	466
	Standard riveted steel pipes		
4	.024	0.87	43.0
6	.023	.86	120
8	.022	.86	244
	Multiplate steel pipes		
6	.034	0.91	92.0
8	.033	.90	193
10	.032	.89	339
	Concrete box culverts		
4x4	.015	0.88	75.1
6x6	.015	.87	192
8x8	.015	.86	363
10x10	.015	.86	588

[1]All culverts are assumed to be 250 feet long.

According to John Cook of Charleston CPW (oral commun., 1992), the quality of water at the site W4 intake is nearly equivalent to the quality of the water in Durham Canal under "normal" site W4 withdrawals. Therefore, it may be assumed that any portion of Bushy Park Reservoir flushed at the same rate will be of the same quality. The target days-to-flush at the site W4 flushing rate, shown in figures 16 to 19, were computed by the equation:

$$F_{CPW} = \frac{F_{W4}\ V_{CPW},}{V_{W4}} \tag{5}$$

Where
- F_{CPW} is the number of days for a particle of water to travel from Durham Canal to the CPW intake,
- F_{W4} is the number of days for a particle of water to travel from Durham Canal to the W4 intake,
- V_{CPW} is the volume of water, in cubic feet, stored in the reservoir between Durham Canal and the CPW intake, and
- V_{W4} is the volume of water, in cubic feet, stored in the reservoir between Durham Canal and the W4 intake.

Then, if the CPW intake is at its current location on Foster Creek,
- F_{W4} is 1.2 days,
- V_{CPW} is 3.37×10^8 cubic feet,
- V_{W4} is 0.774×10^8 cubic feet, and
- F_{CPW} is 5.2 days.

If the CPW intake were to be located to site W3 on Back River,
- V_{CPW} is 1.99×10^8 cubic feet, and
- F_{CPW} is 3.1 days.

This target flushing rate may be somewhat high, because adequate flushing could possibly be attained by withdrawals less than the current site W4 flushing rates. The target flushing rate could also be low, because the water quality could degrade more as it travels over more aquatic growth on its way to the Charleston CPW withdrawal point. However, in the absence of sophisticated water-quality studies, this estimate should be fairly acceptable.

With 50 Mgal/d withdrawals at site W1 intake and with eight 6-foot pipes in the dam, as shown in figures 16 and 17, the days-to-flush are about twice the estimated target. At 118 to 150 Mgal/d, and the same number of pipes, the days-to-flush exceed the target rate by less than one day. If the intake is at site W3, as shown in figures 18 and 19, the target days-to-flush can be reached for all withdrawal conditions with about six to eight 6-foot culverts.

SUMMARY AND CONCLUSIONS

The Charleston Commissioners of Public Works (CPW) withdraws 65 to 90 percent of the freshwater supply from the Edisto River and 10 to 35 percent from the Foster Creek part of the Bushy Park Reservoir. The reservoir is formed by a dam across the Back River and is supplied with freshwater by Durham Canal, which connects it to the West Branch Cooper River. The West Branch Cooper River is fresh at Durham Canal and brackish at times where it is joined by the Back River below the Bushy Park Dam.

The Bushy Park Reservoir is eutrophic; contains large amounts of aquatic growth; and does not always meet State standards for dissolved oxygen. Foster Creek at times has high concentrations of organic compounds associated with taste and odor problems and high fecal coliform counts, compared to the reservoir. Water from Durham Canal is more economically treatable than either the water of Foster Creek or the Edisto River. It was expected that water quality in Bushy Park Reservoir could be improved by decreasing its retention time. Therefore, the CPW considered ways to decrease the retention time of water in Bushy Park Reservoir. The one-dimensional unsteady-flow model (BRANCH) was used to simulate flows in the Cooper River from Pinopolis Dam to Yellow House Creek and in the Bushy Park Reservoir. The particle-tracking function of the model was used to simulate flushing times in the reservoir. The particular-tracking function of the model computes the distance traveled by a particle of water from velocity and elapsed time; dispersion, diffusion, and decay of contaminants are not considered. Subroutines to compute flow through tide gates were added to the BRANCH model. Full-pipe flows were computed for the tide gates by multiplication of the fall across the tide gates by "K" factors. The "K" factors can be computed for any culvert size or type.

The model was calibrated and verified by using flow measurements at 15 sites and stage data at 17 stations. The mean stage residual ranged from -0.14 to 0.36 ft and the maximum standard deviation of stage residuals was 0.26 ft for calibration and 0.22 ft for verification. Volume errors for calibration and verification for key flow measurements ranged from -19.3 to +13.0 percent. Standard deviation of residuals for the key flow measurements generally were 32.8 percent or less. The model could not be calibrated more accurately within the scope of this project because of effects of abandoned rice fields and dikes on the Cooper River, aquatic growth in the Bushy Park Reservoir, and the general complexity of the large estuarine flow system. Therefore, a sensitivity analysis of flushing time was conducted for a viable scenario: a "K" value of 1,216, a new Charleston CPW intake at site W3, Charleston CPW withdrawal of 50 Mgal/d, and normal withdrawal at site W4 thermoelectric power plant. The sensitivity analysis evaluated changes of ±25 percent to eta values, ±1.00 ft datum correction to Bushy Park Reservoir and Durham Canal, and ±0.50 ft datum correction to stage data at the external boundary stage stations. The sensitivity analysis showed that all results were within ±8 percent, except for ±16 percent when adding ±1.00 ft to the datum of the cross sections in Bushy Park Reservoir and except for -15 percent when lowering the datums of stations 02172065 and 02172033 by 0.5 ft. Therefore, flushing rates for this scenario are fairly insensitive to variations of eta values, datums of cross sections, or stage data at external boundaries.

The project duration was too short to collect sufficient data to quantify the frequency or duration of flows or flushing times. Therefore, the period September 15 to 29, 1991, was selected for scenario computations, because this period experienced the lowest outflows through Bushy Park Dam and consequently the longest flushing times. Full pipe flow was assumed for tide-gate computations with "K" factors of 0, 200, 608, 1,220, and 1,820; zero and normal withdrawals at site W4; Charleston CPW withdrawals of 50, 118, and 150 Mgal/d; and location of CPW intakes at Foster Creek or site W3 were simulated.

The retention time (days-to-flush) of water between Durham Canal and the site W4 intake, attained by current site W4 withdrawal rates, should yield sufficiently improved water quality for treatment. Target flushing rates of 3.1 and 5.2 days were established for CPW intakes located at site W3 and Foster Creek, respectively, by using the site W4 flushing rates. With 50 Mgal/d withdrawals at Foster Creek intake and eight 6-ft concrete pipes at Bushy Park Dam, the days-to-flush are about twice the estimated target. If the withdrawal rate is increased to 118 or 150 Mgal/d for the same number of pipes, the target rate is exceeded by one day. If the Charleston CPW intake is at site W3, the target days-to-flush can be reached by withdrawal rates of 50 to 150 Mgal/d with six to eight 6-ft concrete pipes. Sufficiently improved water quality for use by Charleston CPW from Bushy Park Reservoir is most likely if the intake is located at site W3. Use of safety factors in the design could be used to compensate for the uncertainties in the model.

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CONVERSION FACTORS AND VERTICAL DATUM

Multiply	By	To obtain
mile (mi)	1.609	kilometer
acre	4,047	square meter
square foot (ft^2)	0.09294	square meter
million gallons per day (Mgal/d)	0.04381	cubic meter per second
cubic foot per second (ft^3/s)	0.02832	cubic meters per second
cubic foot	0.02832	cubic meter

Sea level: In this report "sea level" refers to the National Geodetic Vertical Datum 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.