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PROCEEDINGS OF STORMWATER AND WATER QUALITY MODEL USERS GROUP MEETING

January 31-February 1, 1985

Edited by

Thomas O. Barnwell, Jr.
Center for Water Quality Modeling
Environmental Research Laboratory
Athens, GA 30613

ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
ATHENS, GA 30613

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The work described in these papers was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

FOREWORD

A major function of research and development programs is to effectively and expeditiously transfer technology developed by those programs to the user community. A corollary function is to provide for the continuing exchange of information and ideas between researchers and users, and among the users themselves. The Stormwater and Water Quality Model Users Group, sponsored jointly by the U.S. Environmental Protection Agency and Environment Canada/Ontario Ministry of the environment, was established to provide such a forum. The group has recently widened its interests to include models other than the Stormwater Management Model and other aspects of modeling water quality in urban and natural waters. This report, a compendium of papers presented at the users group meeting held on January 31-February 1, 1985, in Gainesville, FL, is published in the interest of disseminating to a wide audience the work of group members.

Rosemarie C. Russo, Ph.D.
Director
Environmental Research Laboratory
Athens, Georgia

ABSTRACT

This proceedings includes 17 papers on topics related to the development and application of computer-based mathematical models for water quantity and quality management. The papers were presented at the semi-annual meeting of the Joint U.S.-Canadian Stormwater and Water Quality Model Users Group held on January 31-February 1, 1985, in Gainesville, Florida.

Application of stormwater management modeling is examined in North American and European settings in several of the papers, including Florida flatlands, North Carolina peatlands, Canadian urban runoff ponds, and Swiss stormwater runoff tanks. Estuary studies reported include hydrodynamic and water quality simulations, phytoplankton-nutrient dynamics modeling, and techniques for assessing reservoir eutrophication. The Hydrological Simulation Program-FORTRAN is the basis for studies of snow melt simulations, of deep pumping effects on surface hydrology, and of phosphorus dynamics in wetlands. Phosphorus concentration data are linked with commonly measured watershed characteristics in another study.

A model is presented to evaluate the cost-effectiveness of best management implementation schemes on two agricultural basins in Florida, and QUAL-II is applied to a resource allocation project in England. In other studies, factor analysis is applied to the management of impoundment water quality, and simulation modeling is used to evaluate aquifer storage recovery in a regional water supply system. An affordable alternative to a mainframe computer environment for continuous modeling is presented.

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STORMWATER MANAGEMENT BY MICROCOMPUTER

B.A. Christensen
Professor, Hydraulic Laboratory
Department of Civil Engineering
University of Florida
Gainesville, FL 32611, USA

A.D. Tilton
Project Engineer
Johnson Engineering, Inc.
Fort Myers, FL 33902, USA

ABSTRACT

Stormwater management and floodplain delineation in very flat areas such as Florida's rapidly developing coastal zones have long depended on input from mainframe computers that may not be directly available to the many consulting engineering offices that attempt to tackle these problems. Technological advances in the microcomputer field that make these tasks possible for owners of the not too expensive latest generation of microcomputers are therefore welcomed by the engineering profession.

The present paper discusses the development and use of a microcomputer oriented program that will yield the necessary information needed by civil engineers and planners dealing with drainage basins of small to intermediate size in the extremely flat areas along Florida's coastline. The program is simpler than similar existing programs for larger computers that take into consideration the many complications that steep or even moderate slopes usually found in drainage basins at other locations dictate.

The program develops simple synthetic hydrographs for all subbasins and routes them through overland sheet flow, channels and man-made or natural reservoirs.

Based on the discharges generated in this way the water surface elevations are predicted by use of the iterative standard step method throughout the system. The depths obtained in this way are applied to improve the routing results in several steps. The initial hydrographs are based on the Manning formula for sheet flow and the assumption that the flat land of the drainage basin serves two functions, namely water storage and conveyance of the stormwater towards the ultimate recipient. Man-made as well as natural watercourses with wide floodplains may be treated.

The method is financially competitive not only during the first floodplain mapping of an area but especially during the following updatings called for by development of the drainage basin.

INTRODUCTION

The need for a simple program that will compute the extent of the floodplain in a flat area for a given rainstorm has evolved over the last ten to fifteen years as population pressures and industrial developments have brought floodplain ordinances and similar legislation into existence. Sophisticated methods for floodplain prediction have long existed and are reflected by such major hydrologic simulation models as the U.S. Army Corps of Engineers HEC programs and the SWMM-program developed for the United States Environmental Protection Agency. These programs usually require mainframe computers for their execution.

The program discussed in this paper is designed to meet the needs of the consulting engineering office without direct access to a mainframe computer. It is well suited for execution on most of the microcomputers that are available today (1985) and is limited to the prediction of the floodplain extent and elevation in a flat watershed of moderate size drained by natural and man-made watercourses. It is considering the overland sheet flow that often prevails in such drainage basins in their natural state.

The program consists of four parts:

A. Computation of synthetic hydrographs for the individual subbasins and composite hydrographs at selected stations. This part includes the routing of hydrographs through channels and reservoirs shown in Figure 1.

B. Computation of depths, elevations of water surface, piezometric head line, energy grade line, and widths of the water surface at all selected stations.

C. Numerical display of results.

D. Graphical display of results.

Since the depths and depth related parameters computed in part B are functions of the discharges found in part A and these discharges again are functions of the depths a major iterative scheme must be involved in the solution. The first set of discharge values is based on an arbitrarily chosen (but realistic) set of depth values. The resulting discharges are used to compute an improved set of depth values which are used for improvement of the discharges in part A and so on in an iterative manner until a satisfactory accuracy is achieved.

Before the program can be initiated the raw data must be presented in a form that makes it compatible with the program. From topo maps of the watershed water divides must be determined, subbasins defined and basin areas, A_B, computed. Stations along the watercourses where ther floodplain elevation and width are to be computed must be defined and the channel cross sections approximated by well defined curves or straight lines. In the following section it is shown how a section with even very wide floodplains may be defined by six numerical values which together with the section's station number will describe the section's location and shape completely.

Rainstorm parameters such as rain intensity i, time of the beginning of the storm t_o, duration T and the geographical extend of the storm must of course also be decided. A runoff coefficient ε taking antecedent conditions into consideration must be evaluated according to the nature and degree of development of the considered subbasin.

The routing of the runoff handled by part A is demonstrated in Figure 1 showing an example with five subbasins. A hydrograph at a certain location is indicated by the symbol HG. The subscript (numbers or a combination of numbers and letters) indicates how that hydrograph is generated.

A single digit (1 through 5) indicates that the hydrograph is a synthetic hydrograph generated for the subbasin having that number. A subscript consisting of two or more digits, but no letters, refer to a hydrograph created by simple superposition of the individual synthetic hydrographs for the subbasins having the numbers referred to by the digits. A letter C in the subscript shows that the hydrograph represented by the preceding symbols of the subscript has been channel routed. Channel routing is carried out by the Muskingum method briefly described by Henderson (1) and in more detail by Overton (2). Similarly reservoir routing is indicated by the letter R.

For example the symbol $HG_{1.2.C.3.4.R.C.5}$ for the hydrograph at the receiving waters in Figure 1 states that this composite hydrograph is obtained by adding the synthetic hydrographs of subbasins Nos. 1 and 2, channel routing the result, adding the synthetic hydrographs of subbasins Nos. 3 and 4, reservoir and channel routing the resulting hydrograph and finally adding the synthetic hydrograph of subbasin No. 5.

Since the program is intended for use in flat coastal areas it is reasonable to assume that all channel and overland flow is subcritical, i.e. that the energy of the flowing waters is predominantly potential rather than kinetic. Consequently, the computation of the backwater profile mentioned in part B must begin at the most downstream control section and proceed in the upstream direction. The standard step method discussed by Chow (3) is chosen. This iterative method determines the water depth and thereby the floodplain elevation at the chosen stations along the watershed's channels and creeks.

DRAINAGE BASIN GEOMETRY

It is most important for a successful use of the program that water divides and cross sections of the watershed's channels are well defined. While the first are found manually from topographic maps the latter may be accomplished by approximating the surveyed cross sections by straight lines and or exponential curves.

An arbitrary cross section is shown in Figure 2. It is divided into three parts, a central part of width b and constant depth d, and a left (subscript 1) and right (subscript 2) bank part. The areas of the two bank parts, $A_{B.1}$ and $A_{B.2}$, may be determined for a given depth from surveys in the field. Using the x,y coordinate systems shown in Figure 2 the two bank part may be approximated by the exponential curves

$$y = a_1 x^{n_1} \qquad (1) \quad \text{and} \quad y = a_x x^{n_2} \qquad (2)$$

where a_1, a_2 = constants and n_1, n_2 = constant exponents to be determined from field data, requiring same width of water surface, depth and cross sectional area of section defined by Equations (1) and (2) and the actual section.

Using this concept the area of the section shown in Figure 2 may be expressed as a function of depth d and known parameters by

$$\boxed{A = bd + \frac{n_1}{1 + n_1} \cdot \frac{1}{a_1^{1/n_1}} \cdot d^{\frac{1 + n_1}{n_1}} + \frac{n_2}{1 + n_2} \cdot \frac{1}{a_2^{1/n_2}} \cdot d^{\frac{1 + n_2}{n_2}}} \qquad (3)$$

HYDROGRAPHS OF SUBBASINS

A synthetic hydrograph for a subbasin may be generated by recalling that the area of that basin has two functions, storage of water and conveyance of water.

Separating these two functions as indicated in Figure 3 and assuming that the flow representing the conveyance of water away from the basin towards its principal watercourse is uniform with depth d_o the differential equation representing conservation of mass may be written

$$\varepsilon i\, A_B = A_B \frac{dh}{dt} + Q = A_B \frac{dh}{dt} + 2L\, d_o\, M\, d_o^{2/3}\, S_b^{1/2} \qquad (4)$$

in which ε = runoff coefficient taking into consideration infiltration and other losses such as evapotranspiration, i = intensity of rain, A_B = basin area, h = elevation of water surface in horizontal part of basin, t = time and Q = rate of outflow.

Assuming that $h \cong d_o$, the Manning formula is used in Equation (4) to evaluate the rate of outflow term using the following symbols: L = maximum length of subbasin, $M = 8.25 \sqrt{g}/k^{1/6}$, g = acceleration due to gravity, k = equivalent sand roughness of channel bed and submerged land surfaces, and $S_b = \sin \beta$ = slope of land where β = inclination of land with respect to horizontal.

Using the boundary conditions that Q = 0 at time $t = t_o$, that the rainstorm start at that time and the rain intensity i remains constant after that time Equation (4) has the solution

$$t = \frac{3}{5} \cdot \frac{A_B}{(2LM \sqrt{S_b})^{3/5}} \cdot \int_0^Q \frac{dQ}{\varepsilon i\, A_B\, Q^{2/5} - Q^{7/5}} + t_o \qquad (5)$$

This may be approximated by a straight line with good accuracy in a t,Q-coordinate system.

Letting T denote the duration of the rainstorm and Q_{max} the maximum rate of outflow at $t = t_o + T$ the equation of that line may be written

$$Q = Q_{max} \cdot \frac{t - t_o}{T} \quad \text{......(6)}$$

where Q_{max} of course is unknown.

After the rain has stopped at time $t_o + T$ the solution of Equation (4) may be written

$$Q = \frac{1}{\left\{\frac{t - (t_o + T)}{K} + \frac{1}{Q_{max}^{2/5}}\right\}^{5/2}} \quad \text{......(7)}$$

where the constant K is given by

$$\boxed{K = \frac{3}{2} \; \frac{A_B}{(2LM \sqrt{S_b})^{3/5}}} \quad \text{......(8)}$$

Equations (6) and (7) represent the rising and falling banks of the synthetic hydrograph, respectively.

To satisfy continuity it must now be required that

$$\int_{t_o}^{+\infty} Q \, dt = \varepsilon i \, A_B \, T, \quad \text{......(9)}$$

an equation that may be used for determination of Q_{max}.

Introduction of Equations (6) and (7) for Q at times $t_o < t < t_o + T$ and $t_o + T < t$, respectively, in Equation (9) and integrating yields

$$\varepsilon i \, A_B \, T = \frac{1}{2} T \, Q_{max} + \frac{2}{3} K \, Q_{max}^{3/5} \quad \text{......(10)}$$

in which the first term on the right hand side may be shown to be predominant.

Consequently Q_{max} may be found from

$$\boxed{Q_{max} = \varepsilon i \, A_B \cdot \frac{1}{\frac{1}{2} + \frac{2}{3} \frac{K}{T} \cdot \frac{1}{Q_{max}^{2/5}}}} \quad \text{......(11)}$$

Equation (11) is to be solved by an iterative process for which a subroutine is to be included in the program.

To include the effect of the "average" preceding storm condition a base flow rate of one tenth of Q_{max} is added to the discharges given by Equations (6) and (7) resulting in the final equations

$$\boxed{Q = Q_{max} \cdot \frac{t - t_o + 0.1\,T}{T}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

and

$$\boxed{Q = \frac{1}{\left\{\frac{t - (t_o + T)}{K} + \frac{1}{Q_{max}^{2/5}}\right\}^{5/2}} + \frac{Q_{max}}{10}} \quad \ldots\ldots\ldots\ldots\ldots (13)$$

for the rising and falling limbs of the hydrograph, respectively. K is found from Equation (8) and Q_{max} by the iterative process outlined by Equation (11).

Figure 4 shows an example of the complete synthetic hydrograph for a subbasin.

ROUTING OF RUNOFF

The hydrographs are channel and reservoir routed through the system as indicated by the example of Figure 1.

The programs subroutine for channel routing is based on the classic Muskingum method.

Reservoir routing is carried out by the storage indication method also referred to as the modified plus method.

COMPUTATION OF BACKWATER CURVES

The computation of depths and thereby elevations of water surface and energy grade line at the chosen stations is based on the standard step method, an iterative method giving the depths at given locations in open water courses with steady or quasi-steady flow. In the flat regions considered here it is indeed a reasonable assumption to assume quasi-steady flow. It may furthermore be assumed that the flow is subcritical, i.e., that the predominant form of energy in the flowing water is potential rather than kinetic. A consequence of the latter assumption is that all step by step integrations of the differential equation describing the free water surface must be carried out in the upstream direction.

Using the symbols of Figure 5 and applying the energy equation from station No. j+1 to station No. j

$$z_{b.j+1} + d_{j+1} + \alpha \frac{Q^2}{2g\ A_{j+1}^2} = z_{b.j} + d_j + \alpha \frac{Q^2}{2g\ A_j^2} + \frac{S_j + S_{j+1}}{2} \cdot \Delta\ell_{j.j+1} \quad \ldots\ldots(14)$$

in which S = slope of the energy grade line at the station indicated by the subscript, and $\Delta\ell_{j.j+1}$ = step length.

The energy coefficient α is assumed constant along the entire watercourse and the areas A_j and A_{j+1} may be found as functions of the corresponding depths from Equation (3).

Equation (14) is now solved for the second term, d_{j+1}, on the left hand side giving the iterative formula for d_{j+1}

$$\boxed{d_{j+1} = d_j + z_{b.j} - z_{b.j+1} + \frac{\alpha\ Q^2}{2g}\left(\frac{1}{A_j^2} - \frac{1}{A_{j+1}^2}\right) + \frac{S_j + S_{j+1}}{2}\ \Delta\ell_{j.j+1}} \ldots(15)$$

Under normal circumstances this equation provides a rapidly converging solution. The first estimate of d_{j+1} is d_j.

The slopes, S, of the energy grade line in stations Nos. j and j+1 may be found from the Manning formula as functions of the corresponding depths,

$$S = \frac{Q^2}{MA^2\ \overline{R}^{4/3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(16)$$

where $M = 8.25\ \sqrt{g}/k^{1/6}$ now must be based on the roughness of the considered channel and A may be found from Equation (9) for the depths d_j and d_{j+1}.

Instead of the usually applied hydraulic radius a resistance radius $\overline{R}$ is introduced in Equation (16) to take the true shear stress distribution along the wetted perimeter of the combined channel and floodplain into consideration. While the use of the hydraulic radius implies a constant bed shear stress along the entire wet perimeter of an open channel, the resistance radius is intended to consider the influence of a much more realistic bed shear stress distribution where the local bed shear stress is proportional to the local vertical depth, i.e., proportional to d-y in Figure 2. This is of special importance in the very wide relatively shallow sections encountered during flooding of the flat lands of Florida's coastal zone. As indicated by the Danish Hydraulic Institute (4) the resistance radius may be written

$$\overline{R} = \left[\frac{1}{A}\int_0^B (d-y)^{3/2}\ dx\right]^2 \quad \ldots\ldots\ldots\ldots\ldots(17)$$

for wide flat sections.

Applied to the section shown in Figure 2, Equation 17 yields the resistance radius $\overline{R}$ for that section

$$\bar{R} = \underbrace{\frac{\frac{n_1}{1+n_1}\cdot\left(\frac{d}{a_1}\right)^{1/n_1} + b + \frac{n_2}{1+n_2}\left(\frac{d}{a_2}\right)^{1/n_2}}{\left(\frac{d}{a_1}\right)^{1/n_1} + b + \left(\frac{d}{a_2}\right)^{1/n_2}}\cdot d}_{\text{Mean Depth } d_m}$$

$$\cdot\underbrace{\left[1+\frac{3}{4}\left\{\frac{\left(\frac{n_1^2}{(2n+1)(n_1+1)}\left(\frac{d}{a_1}\right)^{1/n_1}+\frac{b}{2}+\frac{n_2^2}{(2n_2+1)(n_2+1)}\left(\frac{d}{a_2}\right)^{1/n_2}\right)\left(\left(\frac{d}{a_1}\right)^{1/n_1}+b+\left(\frac{d}{a_2}\right)^{1/n_2}\right)}{\left(\frac{n_1}{1+n_1}\left(\frac{d}{a_1}\right)^{1/n_1}+b+\frac{n_2}{1+n_2}\left(\frac{d}{a_2}\right)^{1/n_2}\right)^2}-\frac{1}{2}\right\}\right]^2}_{\text{Correction Factor for Nonuniform Bed Shear Stress}}$$

......(18)

Use of this expression for $\bar{R}$ and Equation (3) for the cross sectional area in Equation (16) gives the values of the slope of the energy grade line needed for the iterative determination of the depth d_{j+1} from Equation (15).

CONTROL STRUCTURES

The presence of control structures such as culverts, weirs and gates in the watercourse is handled by either computing the upstream water surface elevation from the downstream water surface elevation in cases of submerged structures or introducing a new control section upstream of the not submerged structures.

In the not submerged cases the depth in the upstream control section must be independent of the downstream depth and may be determined from the discharge and the general geometry of the structure.

Culverts, weirs and gates are considered in the model using the commonly accepted formulas for the discharge-head relationships for such structures.

CONCLUSION

The program developed in this paper is well suited for floodplain delineations in small to moderately sized coastal watersheds. In spite of the many simplifying assumptions and shortcuts made the method has proven to give reliable results.

The final display of results include printouts and plots of the energy grade line, the water surface and the channel inverts along the canals and natural watercourses in the watershed. Also the width of the floodplain is plotted along the major watercourses.

Hydrographs may also be plotted at selected stations if it is desired.

A drainage basin with 100 to 200 stations may be handled by most advanced microcomputers equipped with one hard disk drive.

ACKNOWLEDGEMENTS

This research and development effort was sponsored by the Engineering and Industrial Experiment Station of the University of Florida and by Johnson Engineering, Inc., of Fort Myers, Florida. This support is gratefully acknowledged.

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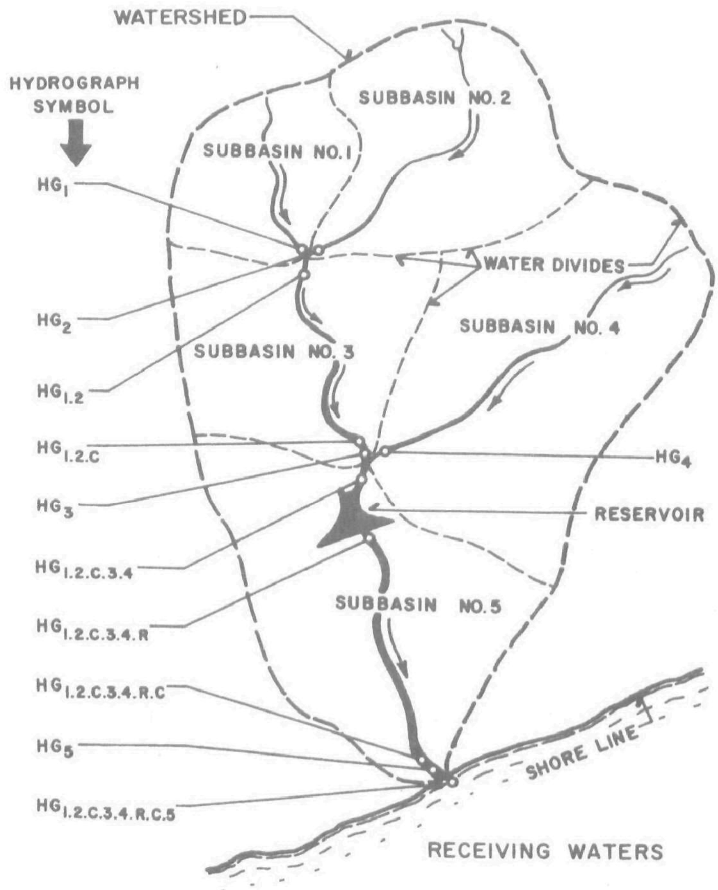

Figure 1. General Lay-Out of Drainage Basin. Routing Scheme.

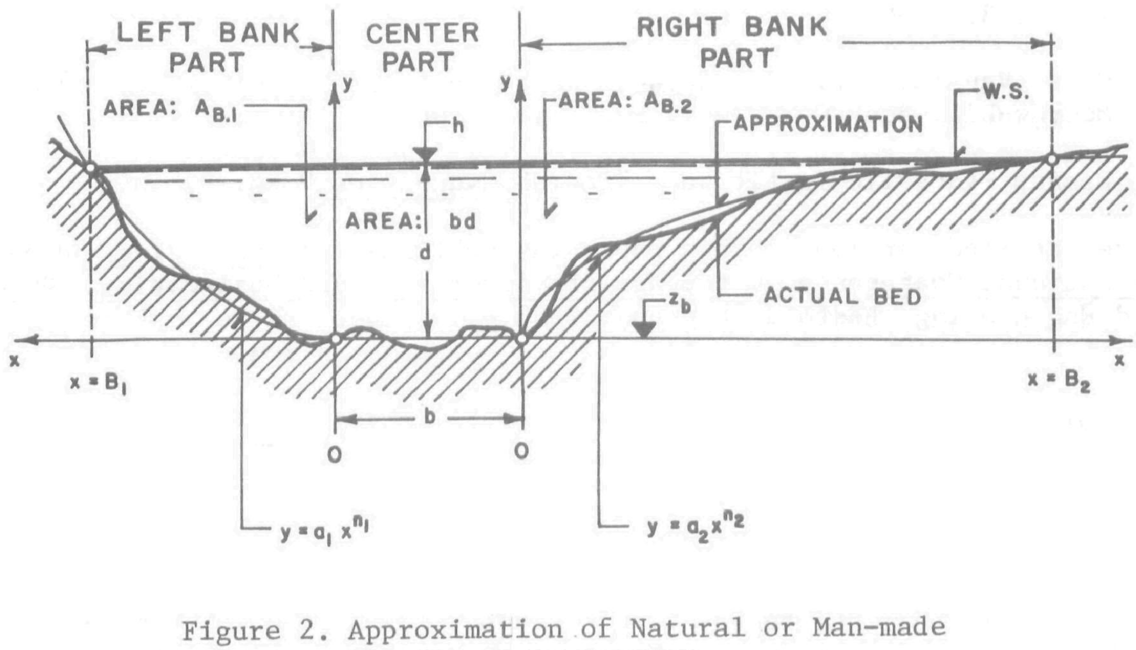

Figure 2. Approximation of Natural or Man-made Channel Cross Section.

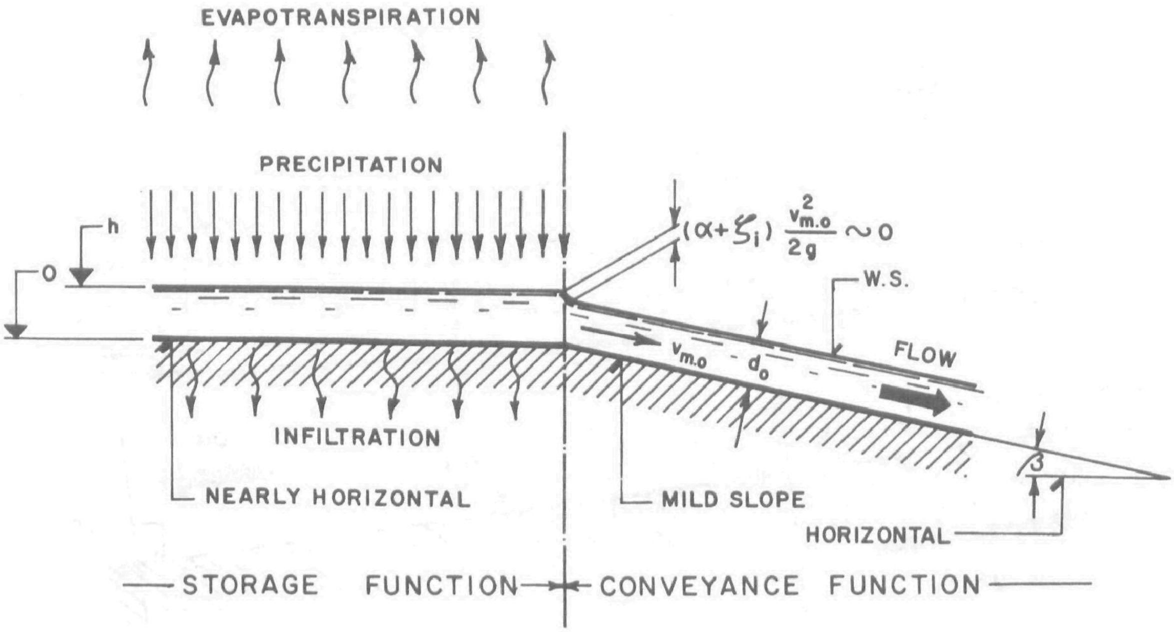

Figure 3. Generation of Synthetic Hydrograph for a Subbasin.

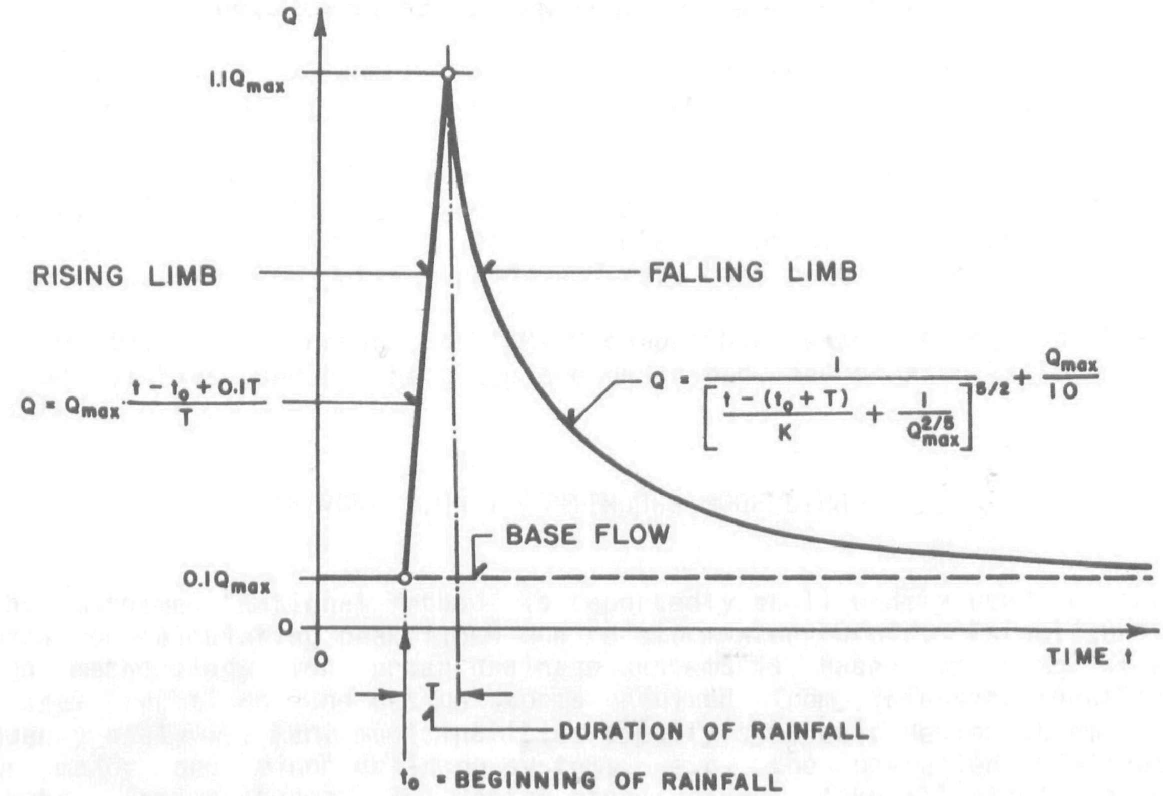

Figure 4. Synthetic Hydrograph for a Subbasin.

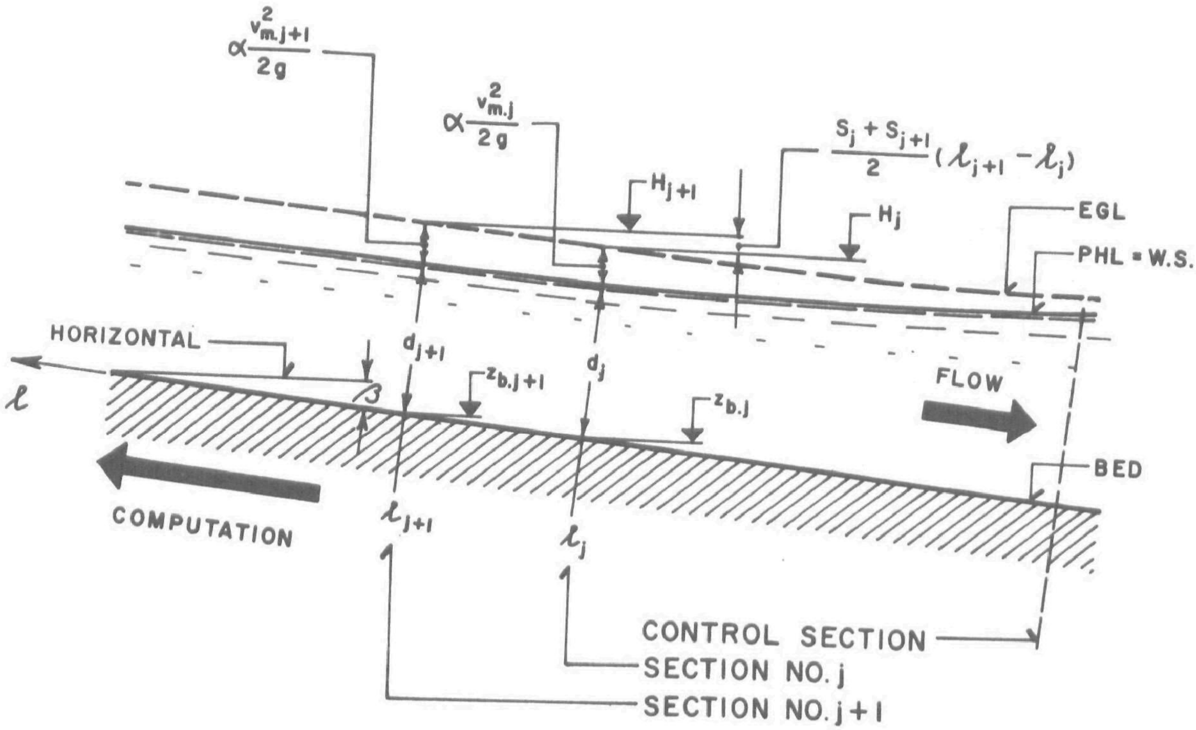

Figure 5. Scheme for Backwater Curve Prediction

AN AFFORDABLE ALTERNATIVE TO A MAINFRAME COMPUTER ENVIRONMENT FOR CONTINUOUS MODELLING

by: W. James and Mark Robinson
Computational Hydraulics Group
McMaster University
Hamilton, Ontario Canada L8S 4L7
Telephone: 416/527-6944

ABSTRACT

Continuous modelling is necessary for reliable estimates of frequencies of flow and pollutional events. When stormwater storage or diversions are considered, the frequencies ought to be especially carefully computed. The advantages of continuous modelling in microcomputer control of combined sewer networks are reviewed. Related activities of the Computational Hydraulics Group are also briefly covered.

Types and sources of data and special time series management software necessary for continuous modelling are mentioned. Some general specifications are suggested for suitable hardware, based on computer storage capacities, computing speeds and data base requirements.

The local area network of IBM-PC compatibles with 9-track and hard-disk capabilities used by the group are described, and pleasantly low costs are cited.

INTRODUCTION - CONTINUOUS MODELLING

The misnamed "Rational Method" is reportedly still widely used in North America for calculating peak flows due to stormwater runoff. Established design methodology for urban drainage systems is based on simplistic, empirical relations and design storms derived from intensity-duration-frequency analysis. Many municipalities specify synthetic design storms for their major and minor drainage systems, e.g. the so-called "Chicago" synthetic design storms. The design storm concept, like all single event modelling, is characterized by the drawback that critical starting values of field moisture and storage levels are unknown.

The proponents of the design storm concept argue on the basis of:

1. Consistency. The technique is traditional practice.

2. Conservative results. Because the method computes excessive peak flows it is inherently safe.

3. Economy. The method is inexpensive and widely held to be quick and easy to use.

The opponents' arguments include:

1. Irrelevance. The concept has not been proven correct. Marsalek (1981, 1977) has shown that design-storm-derived peak flows can be significantly larger than those obtained for historical storms of equivalent probability.

2. Unreliability. No reliable probability can be assigned to the runoff from a single rainfall event (Linsley, 1974).

3. Start-up Probabilities. Current practice specifies a probability for the design storm event period but does not specify the joint frequency of the antecedent dry period or the spatial and kinematic variability of thunderstorm-type rainfall.

4. Inaccurate Input. Design storms do not properly represent long duration, high volume events, which are significant in detention/retention basin design.

5. Pollution Control. On the other hand, the most critical periods for a river or receiving water may occur during dry-weather, low flow conditions, due to the shock loads produced by intense, short-duration thunderstorm activity (Medina, 1979; McConnell, 1980; Sullivan, 1977).

6. Benefit/lost Analysis. Facilities designed for low frequency events will likely prove to be uneconomical when construction and land acquisition costs are compared with the benefits resulting from reduced loadings. Facilities designed for higher frequency events may actually provide a more economical reduction in total annual loading. The design storm concept does not provide sufficient probability information to make this assessment.

7. Effect of the Design Itself. The provision of storage (or other works) changes the response time of the catchment. Thus a different, longer design storm duration is required for the as-built storage system, than the original duration specified. The opposite is true for drainage improvements. This results in an indeterminate flood frequency for the altered drainage system, even if it were correct to transpose the design storm frequency.

It is essential therefore to account for seasonal hydrologic and meteorologic variability in designing or evaluating a water quantity control or water quality management scheme (Sullivan, Heaney et al. 1977; Shubinski, 1980; Donigan, 1980; Shapiro et al. 1980; Leclerc, 1979; Walesh and Syder, 1979; Sullivan, Heaney et al. 1978; Litwin et al. 1981; Walesh, 1979; Barlamont and Van Langenhove, 1981). These requirements can be met by using a properly calibrated continuous model which processes a continuous precipitation record into continuous hydrologic and pollutant loading time series'. Other statistical methods also require a long-term record (Kummler et al. 1981; Shapiro et al. 1980; LeClerc, 1979; Heaney et al. 1978; Medina and Buzun, 1981; DiToro and Small, 1979).

Continuous modelling requires as input long records of hydrometeorological data such as rainfall, evaporation, temperature and streamflow. In Canada, the data are available from government agencies such as the Atmospheric Environment Service and Water Survey of Canada or from private groups such as our Computational Hydraulics Group (CHG). The information is typically distributed on 9-track magnetic tapes since the amount of data required can be voluminous. For example, an eight year record of hourly rainfall intensity at the Mount Hope Airport station in Hamilton consists of approximately 9800 records.

Depending on the situation being analysed, long-term continuous modelling can generate even larger quantities of output time series. This volume of data requires special statistical analyses and graphical displays to be properly understood. As well, archiving of results is often a study requirement. It should be evident that input and output time series data management can be difficult.

Continuous urban hydrologic models such as SWMM are constrained to using only a single station record of rainfall intensity and a constant computational timestep. This situation has probably resulted from the difficulties associated with managing multiple long-term hydrometeorological data sets. Special software is essential for I/O time series management. This software should be capable of retrieving data from and archiving data on magnetic storage media, interfacing this data with an appropriate model package, statistically analysing output time series' and graphically displaying the model output.

During the course of this study a data base management system, CHGTSM, was developed by the CHG. The system has been implemented on an IBM-PC compatible system of microcomputers using a 52-megabyte hard disk and 9-track tape backup. The single data base is coherent and available to every member of the modelling group. The data base currently contains all rainfall, streamflow and pollutant loading data collected by the CHG in the City of Hamilton for the period 1980 to 1984. CHGTSM is capable of aggregating and disaggregating rainfall intensity time series', over variable timesteps for multiple gauging locations. CHGTSM is felt to be an essential element in continuous modelling using stormwater models such as SWMM3.

INTEGRATED SWMM SOFTWARE

Inexpensive microcomputers permit new approaches to the problems of pollution and flooding arising when intense storms track across metropolitan or industrial areas, dumping polluted water into rivers, harbours, etc.

The necessary techniques include:

(a) Field instrumentation,
(b) Data capture and management,
(c) Atmospheric fallout, pollution build-up and wash-off models,
(d) Storm dynamics modelling,
(e) Large scale continuous modelling of urban runoff,
(f) Algorithms for unsteady flow in complex networks,
(g) Receiving water models,
(h) Statistical post-processing and modelling, and
(i) Real-time control.

There is a critical need for measuring rainfall intensity and stormwater discharge accurately in urban areas. This requires high spatial and time resolution, typically at one-minute intervals. To meet this need, rainfall intensity monitoring equipment was developed by CHG, based on single-chip microcomputers utilizing reusable cassette tape as a recording medium. The equipment is currently in use in Hamilton (14 gauges), in Ontario as part of the acid precipitation network (6 gauges), in the Arctic (3 gauges), the Ottawa area (6 gauges), Oslo (6 gauges) and Kentucky. Software (TRANSPLOT) was developed to process the data and to archive it on tape.

An important interdisciplinary research topic is the relation of atmospheric pollution to stormwater modelling. Fallout from industry and vehicular traffic is thought to be a major contributor to surface pollutant loadings in large metropolitan areas. An attempt was made by our group to identify the sources of pollutants available for washoff, and the mechanisms of build-up and washoff. The computer program is known as ATMDST.

The development of appropriate software to determine the movement of stormwater pollutants through a city-wide network of combined sewers is a closely related problem. This software processes information on the dates of street cleaning activities and incorporates algorithms for processes such as traffic, interception by the leaf canopy, and so on. The program is called CHGQUAL.

A computer program package, RAINPAK, has been developed for simulating storm dynamics. Variations in storm speed and direction were found to produce storm flows and pollutant concentrations significantly different from the conventional assumptions of stationary storm distributions. The model accounts for temporal and spatial variations in storms, such as ageing, merging, splitting of storm cells, and so on. Analysis of data from rain sensors is performed by the programs STOVEL, THOR3D, THOR4D, and THOR4DPT, in RAINPAK.

A computer model, OVRFLO3, was developed for modelling sideweir diversion structures. This was motivated by the need to accurately model the first flush loadings from urban areas which will reach the treatment facility, or be diverted to the outfalls.

A computer model, TOTSED, was developed to predict bed sediment and suspended sediment load as a function of time and distance along a one-dimensional quasi-steady-state receiving area near a combined sewer outfall. The model was calibrated using data obtained through a sampling program carried out in the Chedoke Creek outfall channel in Coote's Paradise in Hamilton. TOTSED provides an interface between an urban drainage network and a receiving water body.

In order to facilitate the use of thse models in a teaching environment, a series of pre-processing programs has been developed. These programs, FASTSWSMM3, FASTHEC2 and FASTHYMO, prompt the user for input data in the appropriate sequence, to which he responds by providing data in free-format and/or optional commands which direct the job path along various routes. The pre-processors take care of all system job control language, design file manipulation, and sequentially execute the programs. In this way the user can focus on the hydrologic problems without having to be concerned about the computer system, thereby reducing time expended on learning and/or carrying out calibration, validation or sensitivity analyses. The system has been adapted to IBM-PC compatibles and is known as PCSWMM3. The system is depicted in Figure 1.

The models ATMDST, CHGQUAL, RAINPAK, OVRFLO and TOTSED were designed to be incorporated as additional "blocks" of SWMM3. Each will be incorporated in PCSWMM3.

A field program has been an essential part of the research. The activities were generally as follows:

a. Install and maintain flow gauges at urban streams, overflow structures and outfalls,

b. Install and maintain rainfall intensity recording networks in metropolitan areas,

c. Collect stormwater samples at flowgauging locations for selected storm events at approximately five minute intervals. These samples were subsequently analyzed for suspended solids, BOD5, nitrogen and phosphorous, as well as, occasionally, other constituents.

Using observed event rainfall intensity, discharge and pollutant concentration data, obtained from the field program, a discrete event model of the Hamilton urban drainage system was constructed, calibrated and validated. The RUNOFF Block of SWMM3 was used for this purpose. The basic time-step for the general model is five minutes. A coarse model of the system (time-step of one hour) was also developed; all diversion structures were assumed to be operating such that flow was directed to the receiving waters.

The coarse model of the drainage system was run continuously for a period of nine years (May to October inclusive) at a time-step of one hour using rainfall data from the Environment Canada Archives, in order to develop long-term loadings of pollutants to the Hamilton receiving waters. Special routines (DATANAL) were written to interface with these data tapes.

The results of the continuous modelling were subjected to statistical analyses, in order to develop "easy-to-evaluate" equations for predicting stormwater pollutant loadings for interfacing with a model of the Hamilton Harbour.

To lessen the impact of pollutant loadings in the receiving waters, due to sewer overflows, real-time control by microcomputers located in diversion structures and in storage tanks should be considered. A detailed study of the design of a microprocessor circuit for installation in a specific, existing overflow structure was conducted. The software is called RTCONTROL. This instrumentation integrates with the rainfall and discharge monitoring equipment mentioned earlier to control overflows and make maximum use of in-system storage.

EXISTING HYDROMETEOROLOGIC DATABASES

The need for improved access to data and information has been widely recognized. As a result, electronic data bases have become an essential

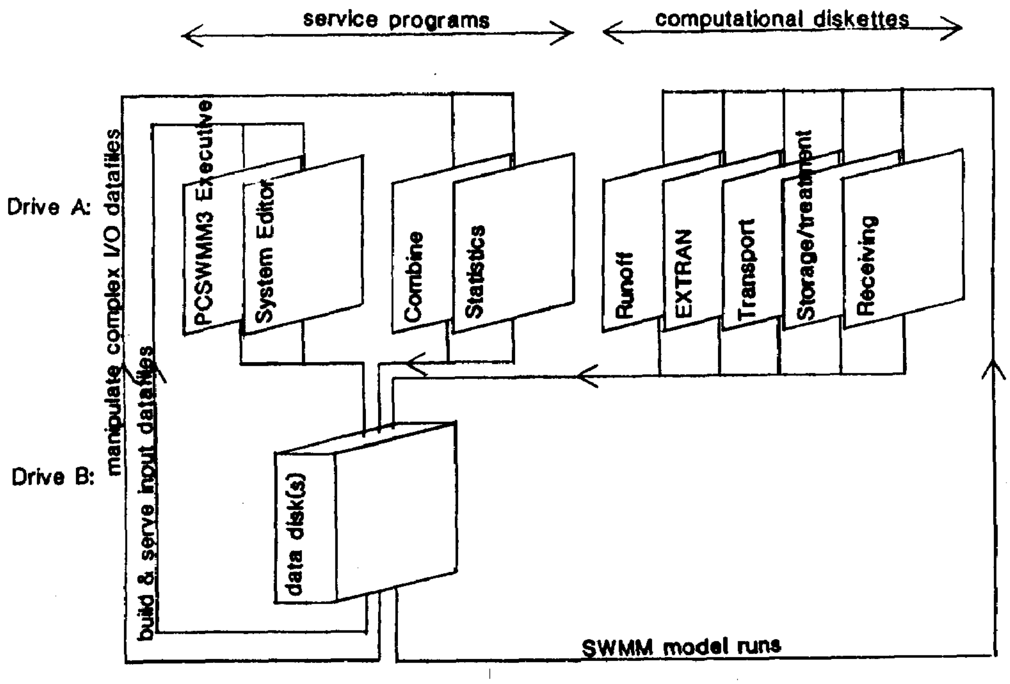

FIGURE 1

tool to fully disseminate or use available information. There are more than 2,000 data bases in North America, which anyone with a microcomputer or terminal, modem and the appropriate accounts and passwords, can use. The main objective of an electronic data base is not so much the provision of information but access to that information.

The distributor of the data bases is the agency that runs the mainframe computers in which the data bases are stored. Sophisticated software makes it possible to find the required information within seconds. One of the larger distributors, for example, is Dialog Information Services, a subsidiary of Lockheed Corp. of Burbank, California.

In the U.S., climate related information data bases are managed by the National Oceanic and Atmospheric Administration (NOAA). The NOAA's "National Environmental Satellite, Data and Information Service" (NESDIS) established, the National Environmental Data Referral Service (NEDRES) in 1981.

The NEDRES, of which Canada is one of the many international data contributors, is the nerve centre of a comprehensive climate data archive. The centre assists in determining the existence, characteristics, content, location and conditions of data files. It does not supply the data, but refers the user to the data holder. Detailed inquiries about NEDRES can be directed to the following address:

National Environmental Data Referral Service Office
National Oceanic and Atmospheric Administration
3300 Whitehaven ST. N. W.
Washington, D.C.
(202) 634-7722 (FTS 634-7722)

With reference to climatic data for Canada, most of the information can be obtained directly from the various divisions of The Atmospheric Environmental Service (AES) in Toronto. The provision of data involves mainly the regional climate units and the Canadian Climate Centre (CCC) at AES. Besides the provision of hard copies and magnetic tapes, CCC offers direct access to the central digital archive. Qualified users can obtain the necessary data and undertake a variety of data management functions and manipulations.

In Canada and Ontario some other data providers are:

Water Survey of Canada (WSC)
Canadian Hydrographic Service (CHS)
Ontario Ministry of the Environment (MOE)
Ontario Ministry of Natural Resources (MNR)
Ontario Hydro

The type of data collected by the above agencies is determined mainly by their given mandates or specific needs. The more frequent data include:

- hydrometric data (water levels and flows)
- meteorological data

- snow surveys
- water temperatures
- ice cover

In addition, information on sediment and water quality is available for a limited number of stations.

In the Canadian National Climatological Archive, managed by CCC, the data are retained in the following three forms:

PAPER: The storage of data on paper was for many years the only way of preserving data and covers a period of operation as far back as the mid-nineteenth century. The data are published in serial publications which, for completeness, are summarized below:

a) Climate Perspectives, contain a weekly summary of national weather events,
b) Monthly Meteorological Summaries, contain hourly and daily meteorological data including monthly means and extremes,
c) Canadian Weather Review, permits a preliminary review of monthly weather based on unverified data,
d) Monthly Record of Meteorological Observations, a comprehensive publication of verified meteorological data,
e) Monthly Radiation Summary, contains hourly radiation data,
f) Annual Meteorological Summaries, contain a short historical account of the stations and the meteorological data,
g) Supplementary Precipitation Data, contains listings for long-duration recording precipitation gauge stations, daily and maximum amounts from precipitation gauges, and a summary of storage gauge records, and
h) Snow Cover Data, contains data from over 1800 snow courses operated across Canada.

In addition to the publications mentioned above, various studies as well as a comprehensive package of climatic data normals are available.

WSC supplies various publications:

a) Surface Water Data Reference Index, contains descriptive information for all gauging stations,
b) Surface Water Data, contains daily discharge and daily water level data for rivers and lakes,
c) Historical Streamflow Summary, contains a summary of monthly and annual mean discharges, and annual extremes and total discharges,
d) Historical Water Levels Summary, contains a summary of monthly and annual mean water levels and annual extremes,
e) Sediment Data Reference Index, contains descriptive information for sediment stations,
f) Sediment Data for Canadian Rivers, contains sediment data, including daily suspended sediment concentration and suspended load, and particle size distribution,

g) Historical Sediment Data Summary, contains summary of monthly and annual mean suspended sediment load, annual extremes of suspended sediment concentration and suspended load, and annual total suspended load.

MICROGRAPHIC: The storage of meteorological data on microfilm began in the 1940's and today almost all source documents are retained on microfilm. Besides 12,000 reels of microfilm, the National Climatological Archive consists of over 10,000 microfiche, containing long-term abstracts of meteorological data up to 1980, organized chronologically by station.

WSC recently started to provide hydrometric data and sediment data on microfiche. The hydrometric data include daily streamflow and water level data prior to 1979. The sediment data cover periods prior to 1978.

DIGITAL: The storage of data in machine-processible form is relatively new and began in 1950 when 80-column punch cards were introduced for data storage. In the mid-sixties, the storage of data was transferred to magnetic tape.

Today, the digital archive contains quality controlled data from all stations. In addition to the station observations of meteorological data, digital precipitation observations at 15 minute intervals collected by six digital precipitation radars (SCEPTRE network) are available (magnetic tape, 1600 bpi).

The use of the digital archive is facilitated by numerous computer programs developed by CCC. The standard archival formats permit extraction of specific parameters from the archival file.

Hydrometric and sediment data are available from WSC in computer-compatible form. Usually, the data are supplied on magnetic tape, written in EBCDIC (odd parity) on 9-track at a density of 1600 bpi.

CHG-TIME SERIES STORAGE SYSTEM

When several users require operational data which is being updated, and which is so large that individual copies would usurp a significant share of their computer resources, centralized control of the data bases is advisable. This avoids redundancy and inconsistency, and encourages standards, security, integrity and co-operation.

A commercial relational DBMS, designed to organize information into a collection of different attributes with a common relationship, would not be efficient for computational hydrology/hydraulics. The basic element of information in a relational DBMS used to maintain water resources data consists of a data item such as a water level, flow or sediment concentration for a particular station at an instant. A better alternative is to use a block of sequential time series (TS) data as the basic element of information. The basic concept underlying a TS management system (TSM) is the organization of data into records of continuous, applications-related elements, as opposed to individually addressable data items.

Our custom made, time series based, pseudo-relational TSM uses a two layer hierarchial local area network (LAN) of work stations. The time series store (TSS) resides in the main hard disk of the LAN and has a tape back-up. The capacity of the hard disk is equal to the capacity of a "standard" 9-track tape plus sufficient space to store important files. The top level of the hierarchial LAN is a work station which acts as a 'Boss'. The Boss computer holds the TSS in the hard disk and magnetic tape drive. The tape drive is main-frame compatible (9-track reels). The Boss also holds modem connections to external databases. The lower level of the hierarchy comprises several work-stations which hold the hydrologic application packages. The overall data flux between the hard disk and lower level nodes is through the Boss computer, which also holds Distributed Processing Software (DPS), which control the overall process. A user at a node can request only high-level operations, such as opening a file, or writing into a file which will be specified by the Boss computer. Users can never destroy the integrity of the network file system.

Figure 2 is an overview. The nodes and the boss are all IBM-PC compatibles, each having the same basic resources, except the 9-track tape drive, hard disk, modem and printer at the Boss CPU.

CHGTSM is written in ANSI-FORTRAN '77.

A data acquisition system used to load field data into a time series store is designed to automatically collect large amounts of data from one or more sources and store or transmit the data for future or immediate processing. When continuous hydrometeorological data is stored at a fine time resolution, large quantities accumulate very rapidly and it is therefore essential that the data collection be computer compatible.

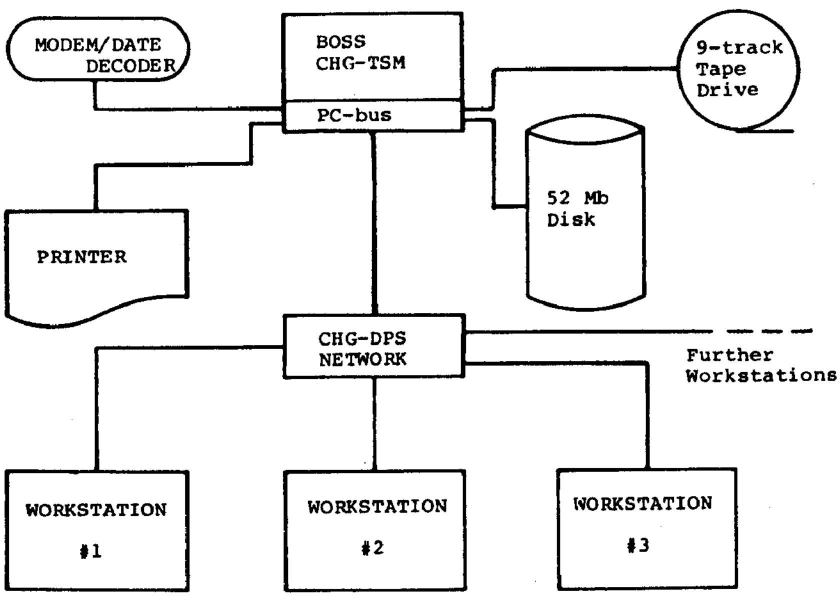

FIGURE 2

Local microcomputer-based data acquisition loggers (DAL), designed to gather data from sensors, e.g. rain gauges, that partially process the data, and store the information on inexpensive media such as magnetic cassette tapes, are used by CHG. The DAL contains one or more acquisition channels, and it is possible to expand the system to include other measurements in the future such as temperature, water conductivity, and pH.

Some of the advantages resulting from the use of a local microcomputer DAL include variable sampling rates, data processing, continuous or intermittent operation, and programmability to cover other data acquisition needs. A microcomputer-based data decoder (DD) is used to automatically retrieve the information stored on tape, verify and transmit it to the base computer. The advantages of this process over manual methods used previously include the prevention of random errors and reduced time.

The uses for which data is intended determine the type of data processing to be performed. For example, the rainfall time series may be processed to compute storm dynamics, produce intensity-duration-frequency curves, tables or isohyetal maps. The data processing should facilitate insertion of the time series into input files to be used by computer program packages, such as PCSWMM3.

Our data acquisition utilities (CHGDAS) - to receive, interpret, store, process, and present the information-currently functions on APPLEIIe and PDP11/23 systems. Two types of processing are carried out: the first is the derivation of a hyetograph for each of the storms and for each monitoring site; the second is input data preparation for computer programs for storm and streamflow modelling. These programs are used in turn to determine peak, average, daily, monthly, and annual amounts of runoff and pollutant. Models are also used to investigate a wide range of design alternatives and strategies for minimizing floods and pollution.

Individual raingauge records are processed to produce hyetographs, plotted at various integration time intervals. Long integration time intervals can be very misleading regarding instantaneous rainfall intensity, if the timestep covers widely varying instantaneous intensities. This kind of data tends to underestimate the short period, high intensity rainfall.

Even a cursory inspection of rainfall distributions illustrates the wide variety of hyetograph shapes. There is no typical rain sequence even though the general sequence might consist of a rapidly increasing rate of rainfall with a maximum intensity reached in the first 15 minutes of a thunderstormon, followed by a period in which intensity decreases to zero or becomes inappreciable. There are a large number of storm characteristics important to local hydrology. Accurate information concerning these characteristics in time and space is indispensable for the cost-effective design of hydraulic structures. Thus, even though previous practice indicates a need for rain data at 1 hour time steps, it is desirable to collect data at (say) 2 minute intervals. Such data are more useful for urban hydrology, such as deriving regional design storms for traditionalists.

HARDWARE FOR COMPUTATIONAL HYDROLOGY

As indicated above areas of fundamental importance to urban hydrology include modelling; microprocessor-based instrumentation; field data acquisition, data management and presentation; data archiving; data base accession; application program systems, incorporating many water disciplines; program maintenance and support; documentation and report generation; text processing; and real-time control of water systems. Application programs include topics in meteorology; hydrology; hydraulics; municipal and environmental engineering, for example, water distribution, pump stations, drainage networks; water and wastewater treatment plants; water quality control; river systems; and limnology.

As illustrated by the CHG software described earlier, activities involved in a typical study include:

1) set up microcomputer-based field instrumentation, typically sampling on a 300 s integration period;

2) acquire a data base of rainfall, runoff, and water quality parameters for the system to be modelled;

3) collect environmental data as required by the models;

4) develop new models and apply continuous models;

5) verify, perform sensitivity analysis, calibrate and validate the continuous models;

6) insert the continuous output time series from the models into the data base system;

7) conduct a statistical analysis of the continuous output time series;

8) develop transfer function or ARIMA models of the continuous input and the output time series;

9) develop control programs for diversion and control structures in the drainage systems being modelled;

10) elaborate the original continuous models to incorporate the new control programs for the control structures;

11) run the continuous models with the control structures appropriately modelled for the full time series record;

12) compare the output for various control structure programs and choose the 'best' control program;

13) summarize the final output;

14) prepare documentation;

15) install the 'best' real-time control system in the field; and

16) set up a continuing program of data acquisition, modelling, and software maintenance to ensure model and prototype performance.

Clearly the data base management system (DBMS) is of central concern.

The ready availability of sophisticated data base management systems, distributed multiprocessor disk operating systems and integrated graphics packages offer a variety of tools that should be considered. Although difficult to estimate, the cost performance of powerful personal microcomputers, with a central hard disk system, and using the new generation of integrated software, is evidently better than that of mainframes, at least for memory requirements up to (say) 4 Megabytes and hard disks up to 400 megabytes. Our new software is aimed at this environment, currently functioning in about one-tenth this space.

A simple but powerful user language is being developed for two types of database: one that handles spatial data, e.g. drainage networks,land use, soil, and vegetation distribution, etc., and another that handles time series data, i.e. hydrological data. The dialogue for the two systems should be designed to be mutually compatible to assist cross reference and intercomparison between the two resource data archives. Input is menu-driven. Output includes tabular daily, weekly, monthly or annual summaries, and graphics.

Commands for the following activities will eventually be included in the dialogue:

1. summarize contents of file
2. summarize contents of batches of data on the file
3. copy raw data onto file
4. add time series data or a rating curve to a file
5. read magnetic tape
6. convert format of ASCII dataset
7. copy an entire batch of data
8. delete an entire batch of data
9. copy any portion of data from one file to another
10. delete any portion of data from a file
11. process event rainfall data
12. transform series data, multiply, add, apply ratings, etc.
13. run user specified model, e.g. SSARR
14. list series data or rating curves
15. plot series data with time
16. print rating table
17. plot rating curves
18. print-plot values and time
19. print monthly/daily/seasonal/annual summary
20. print distribution of values
21. exit from DBMS

At this time our network software has limited capability and is still being developed and implemented.

CONCLUSIONS

The hardware installed by our group is as follows:

				Estimated approx. cost $k
Boss:				
	8088-based portable PC-compatible 512kbytes memory 2 floppy disk drives 8087 co-processor			3
	9-track tape drive interface card software			6
	52 Mbyte hard disk (43 formatted) interface card software			3.6
Modem:				
	1200 BAUD, dial-up			0.6
Printer:				
	High resolution, dot matrix			3
5 Nodes:				
	same as boss CPU	5 x 3	=	15
Software:				
	MS-DOS 2.1	included		
	MS-FORTRAN 3.2			0.3
	Network (allow)			1.1

Total:				29.6
	Approximate cost per station			5.0

Regarding large disk storage, the primary constraints are imposed by MS-DOS, which restricts logical disk volumes to 32 Mbytes. Because of this the data base may need to be segmented.

The PC-compatibles chosen were Corona portables, reputed to contain a large power supply and a large fan. The speed of computation with the 8087 math co-processor has been found to be as fast as one-fifth that of a CDC CYBER 172, as shown in Figure 3. Figure 3 was obtained by programming measurement techniques.

The 9-track drive unit includes an intelligent interface that utilizes a Z-80 microprocessor and proprietary firmware to provide data transfer between a Personal Computer, and an IBM mainframe-compatible 9-track tape drive with an embedded formatter. Supplied by the manufacturer of the hardware is a software interface package, TIP (Tape Interchange Program). Utilizing TIP the user may freely transfer data between Personal Computers and the 9-track tape drive.

The interface occupies a single slot in the PC Bus, and is interfaced directly to the 9-track tape drive via two 50-pin data cables.

The tape subsystem is both self-loading and self-threading for ease of use. The subsystem provides high-speed disk-to-tape transfer at rates of 0.7 megabytes per minute, with up to 42 megabytes of data storage per tape.

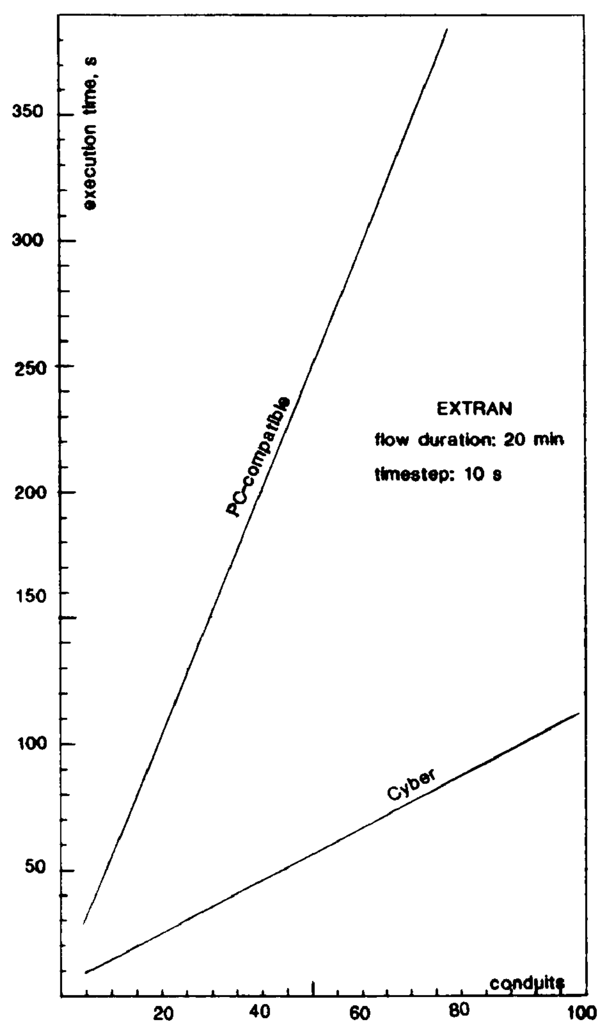

FIGURE 3

The tape unit supports 1600 BPI tapes, with user-specified recording formats from 2 bytes to 16 kilobytes per block. The tape drive will accept all standard 9-track tape drive reel sizes up to 10-1/2 inches in diameter.

Each tape subsystem includes a single density 5-1/4 inch diskette containing three comprehensive programs and two utilities that allow the user to read, write, or inspect a 9-track tape:

1. TREAD - A tape-to-disk-copy/conversion program which will read ANSI compatible data records from the 9-track tape and convert them to DOS compatible ASCII disk records, and store them on disk. This program allows for conversion of data records from EBCDIC to ASCII format, record segmentation of user's choice, and user-specified file name and disk selection.

2. TWRITE - The counterpart of TREAD, TWRITE is a disk-to-copy/conversion program. TWRITE copies DOS compatible disk files to tape in ANSI compatible format. This program allows for ASCII to EBCDIC format, and a user-specified tape record structure.

3. TDUMP - This program allows the user to dump the contents of a 9-track tape to either the user's console or to the system printer. This utility is extremely useful to determine the format in which a tape has been written.

4. TUTIL - This file contains the primary assembly language subroutines that are used in TIP. These subroutines may easily be linked into other 'High Level Language' programs to create user-customized tape control programs.

5. TLINK - A tape control utility module designed to be linked into Basic programs to allow the user to create simple, yet powerful, customized tape control programs.

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MULTIOBJECTIVE DESIGN OF STORMWATER IMPOUNDMENTS

by: Edward A. McBean
University of Waterloo
Waterloo, Ontario, N2L 3G1, Canada

ABSTRACT

Considerable variance in design procedures regarding water quality for stormwater management ponds for control of urban runoff quantities is reviewed, as characterized by several surveys. Also, a simple variable-detention time simulation model is developed to evaluate the water quality behavior of wet ponds. A design procedure is demonstrated through use of multiple objective analysis.

INTRODUCTION

Stormwater management ponds for control of urban runoff quantities have been widely adopted. Nevertheless, there is still considerable variance in design aspects of these ponds and, to an increasing extent, the multiple use character of the ponds is being promoted. Potential multiple uses of the ponds include design for post-development runoff control, for water quality improvement, water surface recreation, for groundwater recharge and for soil erosion control. Conflicts between the design aspects of the ponds often occur, however, since the design requirements for one use are not the same as the design requirements for another use.

In response to the resulting legal and economic concerns arising in part due to the multiple use aspects of the ponds, a number of controversial issues have arisen which involve tradeoffs. Governmental guidelines for stormwater runoff are usually quite subjective in nature, and most designers must therefore interpret and quantify these guidelines using their own judgment, to establish the best design tradeoffs.

As a means of better characterizing current practices, several surveys of practitioners were carried out. The intent of this paper is to discuss some of the findings of these surveys. In addition, a simple variable-detention time simulation model is used to consider the water quality behavior of wet ponds, as a means of demonstrating the use of multiple objective analyses to the problem of design of stormwater ponds. The multiple objective procedure has considerable merit for assisting the decision-making process in demonstrating the nature of the tradeoffs being made.

CURRENT DESIGN PRACTICES OF STORMWATER PONDS

Although the use of stormwater ponds has gained increasing popularity for regulating the quantity and quality of urban stormwater runoff, the number of fully-documented case studies available in the technical literature is limited. As a result, surveys of consultants were undertaken to gain some insight into the current state of the art.

Two questionnaires were distributed, the first to various consulting engineering companies involved in the design of urban drainage works and the second survey to all Canadian cities with a population in excess of 40,000. The responses summarized below represented 32 individual returns associated with one of the surveys and 84 returns associated with the second survey. Collectively, they represent a substantive characterization of current design practices throughout Canada.

The following indicates summary findings:

PEAK FLOW REDUCTION

While all three factors, the increase in the volume of runoff and peak flow runoff rate, and the decrease in lag time, have some influence on downstream flooding it is the magnification of the flood peak that is the single most influential factor in design practices.

The extent by which the peak flow rates are reduced, varies among designers. For the majority of the responses (55%), the capacity of the downstream channel was known (or determined) and the outlets were designed to reduce peak flows to this maximum level. Alternately, when the capacity of the downstream channel was not known, principal outlets were usually designed to reduce all peak flow rates to their pre-development level (27% of responses).

WATER QUALITY CONSIDERATIONS

Stormwater is about equivalent in quality to, or better than, effluent from secondary sewage treatment plants, with the exception of suspended solids concentrations which far exceed the treatment plant effluent levels. It is also recognized that many of the pollutants present in the runoff are in the form of suspended solids; for example, the United States Environmental Protection Agency assumes that 10% of the suspended solids load is BOD (Heaney et al, 1975). It is probably for these reasons that most water quality improvement projects have concentrated solely on the removal of suspended solids as a water quality objective. From the surveys it was found that suspended solids was the only water quality parameter considered in 35% of the cases. (Note that only 41% of the cases used any form of water quality design criteria).

The criterion used to achieve reduced suspended solids levels varies from place to place. In Maryland, sedimentation ponds are required to have a 70% trapping efficiency (removal rate), while a Metro-Toronto Bylaw (#2520) prevents the discharge of stormwater into a natural watercourse when the concentration of suspended solids is in excess of 30 mg/ℓ. These regulations are however exceptional, and most cities have no definite limits on the water quality of urban runoff. Even the Ontario Ministry of the Environment (1978) has stated that remedial measures for the control of pollution due to urban non-point sources are required only if "... they are shown to cause or contribute significantly to violations of the Provincial Water Quality Objectives". As such, the choice of criteria in regard to water quality used for the design of retention facilities is frequently left up to the designer. A pond in Meadowvale, Ontario, was designed to settle out all particles larger than 0.07 mm, while stormwater management criteria for the Professors' Lake Project in Brampton, Ontario, called for the removal of particles greater than 40 microns (David, 1978).

The water quality monitoring schedules used by seven municipalities for in-situ ponds are included in Table 1.

DESIGN AND ANALYSIS OF OUTLET STRUCTURES

Maintaining peak flows at their pre-development level is normally the limiting design requirement, assuming the capacity of the downstream channel is not a more stringent concern. However, the survey results indicated that most of the structures are being designed based on a constant (or maximum) release rate, usually corresponding to the peak pre-development runoff for a particular storm. Such design methodlogies are used by Theil (1977), Grigg (1977), Oscanyan (1975) and Rao (1975). As well, the survey found that 39% of the respondents followed a similar procedure using a peak pre-development flow rate corresponding to a specific storm, most frequently the 25 year pre-development storm.

An outlet designed to meet a specific pre-development peak flow rate will provide little or no peak flow attenuation for events of greater frequency than the design event and will provide greater attenuation for more infrequent events (Lafleur and McBean, 1981). In addition, since urbanization also causes an increase in the volume of runoff, then a specific post-development flow will have a higher return frequency than the pre-development flow of the same magnitude. Such an increase in the frequency of flows combined with higher velocities (resulting from increased peak flows) will cause an increase in the erosion downstream.

There are numerous types of outlets currently being employed, the most popular ones being weirs, orifices and closed conduits (or pipes). In the surveys it was found that 84% of the respondents used one or more of these methods. The remaining respondents used some method of manually-controlled discharge, such as gates, valves or pumps, although current practice is oriented strongly toward making such ponds self-operating.

MULTIPLE USE DESIGN CONSIDERATIONS

The multiple use aspects of the ponds may be divided into primary and secondary functions. In this respect, flood control (13%), water quality control (2%), stormwater management (2%) and cost reduction (1%) were each cited as sole primary reasons for construction of stormwater impoundments. All other cities (81%) which had stormwater impoundments listed at least one

TABLE 1. WATER QUALITY MONITORING SCHEDULES FOR SEVEN MUNICIPALITIES

Water Quality Parameter	City						
	Nepean	Missi-ssauga	Winni-peg	Regina *	Saska-toon	Calgary	Edmonton
B.O.D.5	Weekly	---	5x/yr min	2x/yr*	---	2x/mo	1x/mo
C.O.D.	---		---	---	---	2x/mo	3x/yr
Nutrients	Weekly	4x/yr min	5x/yr min	---	1 yr. after const.	2x/mo	3x/yr
Heavy Metals	Weekly	---	5x/yr min	2x/yr*	---	2x/mo	3x/yr
Fecal Coliform Count	Weekly	4x/yr	10x/yr	2x/yr*	1 yr. after const.	---	1x/mo
Total Coliform Count	Weekly	4x/yr min	10x/yr	---	1 yr. after const.	---	1x/mo
Suspended Solids	Weekly	4x/yr min	5x/yr	2x/yr*	---	---	---
Turbidity	Weekly	4x/yr min	5x/yr	---	1 yr. after const.	2x/mo	---

Remarks pertinent to water quality monitoring schedules.
The above parameters were listed as question response choices while the parameters mentioned below were volunteered under the response choice heading of other parameters tested.

Nepean - Monitoring occurs more frequently if storm occurs.
Winnipeg - Transparency is monitored 10 times a year (10x/yr).
Regina - * Proposed monitoring schedule to be undertaken in the future. Turnover or replacement rate will also be noted twice a year.
Saskatoon - The information in the above table for Saskatoon is the actual schedule of Lakeview pond.
D.O. is also tested a year after construction.
Calgary - Many other parameters are tested.

primary and one secondary use. Table 2 illustrates which secondary functions occur in combination with different primary uses. The figures shown in the table have been collated for each primary use regardless of whether it occurs in conjunction with other primary uses. For example, Gloucester, Ontario, listed flood control and water quality control as the two primary reasons for creating its stormwater pond and aesthetic and open green space as secondary uses for it. In Table 2, Gloucester's secondary use responses of aesthetic and open green space have been recorded twice, once in the flood control row, and once in the water quality control row.

Either on its own, or in conjunction with another primary use, flood control (26%) followed by water quality control (10%) was most frequently cited as a prime reason for impoundment construction. Aesthetic (17%), open green space (15%) and recreation (12%) were the most prevalent secondary uses. In Waterloo the pond was not considered aesthetically pleasing.

TABLE 2. MULTIPLE USE OF STORMWATER PONDS

Primary Use	Secondary Use				
	Aesthetic	Real Estate	Wildlife	Open Green Space	Recreation
Flood Control	10	2	4	13	11
Water Quality Control	4	--	2	6	2
Recreation	2	--	--	1	3
Soil Erosion Control	--	--	2	2	--
Stormwater Management	2	--	1	2	1
Discharge Regulation	2	1	--	1	3
Groundwater Recharge	--	--	1	1	--
Cost Savings	2	2	--	2	3

Remarks
Markham, Mississauga and Winnipeg listed recreation as both a primary and secondary use of stormwater ponds.

EROSION CONTROL

Erosion is most severe immediately downstream of urbanized areas and the degree of erosion diminishes with distance from the urban area. A design parameter involves requirement of wide buffer strips to assist in preservation of natural channels. The wide buffer limits the detrimental impact of man's activities on the banks, such as fill encroachment, garbage dumping, and tree removal. Buffer strips with landscaped lawns and gardens are not equivalent to the natural wooded areas since they do not slow runoff flow as much as natural leaf mulch and do not inhibit access to the channels as would a natural forest bush area.

WATER QUALITY SIMULATION

The design of retention pond facilities for water quality control is not straightforward. The quality of pond effluent is a function of both the influent water quality and the treatment administered to the flow, which is a direct function of the detention time of the water. Both of these parameters vary within the duration of a particular storm event, and it is therefore necessary to identify the naturally occurring combination of these parameters which govern the effluent water quality.

To allow comparisons of the pollutant removal efficiences of alternative pond designs, Lafleur et al (1981) developed a model to simulate retention pond operation. The model is premised on Camp's (1945) clarification theory.

The model employs the concept of "plug flow" which assumes delivery of the flow on a first-in-first-out basis and allows no mixing between plugs. The detention time (t*) of each plug is required for solids removal calculations and so it is imperative that the identity of each plug remains separate and identifiable. The procedure used in the present program to obtain this can be explained using Figure 1. Consider the n^{th} time increment of the inflow hydrograph corresponding to t_1. The cumulative inflow at this time can be found by summing the area under the inflow graph. This can be determined using a finite difference approach:

$$I_{cum\ n} = \sum_{i=1}^{n-1} I_i \Delta t + \frac{1}{2} I_n \Delta t \qquad \text{at } t = t_1 \tag{1}$$

From the assumption of plug flow, it follows that the volume of water contained within any time increment must remain in the pond until the water corresponding to the previous inflows has been discharged. This implies that as the plug (of time increment n) begins to discharge at some time ($t = t_2$) then:

$$O_{cum} = I_{cum\ n} \qquad \text{at } t = t_2 \tag{2}$$

The detention time can then be easily determined since,

$$t^* = t_2 - t_1 \quad (3)$$

If the influent suspended solids concentrations, $C_{in\ i}$, is known for a particular time increment, then the effluent concentration can be found by,

$$C_{out\ i} = (1 - X_{T\ i})\ C_{in\ i} \quad (4)$$

where $X_{T\ i}$ is the average removal rate for time increment i and is a function of:

$$X_{T\ i} = F(H_{avg\ i}\ t^*_i\ v_t\ \alpha) \quad (5)$$

where v_t represents the terminal velocity of the particles, H_{avg} is the average depth over the detention time, and α is a correction factor to account for non-idealized conditions, such as short-circuiting, resuspension of sediments or the non-spherical nature of the particles. It is noteworthy that this simulation model reflects the unsteady nature of the pond's operation.

Since it was found that the variation of particle size distribution with time was small, this parameter was assumed constant throughout the storm. It should also be noted that due to a lack of data, no attempt was made to evaluate α and for current purposes it has been assumed to equal unity.

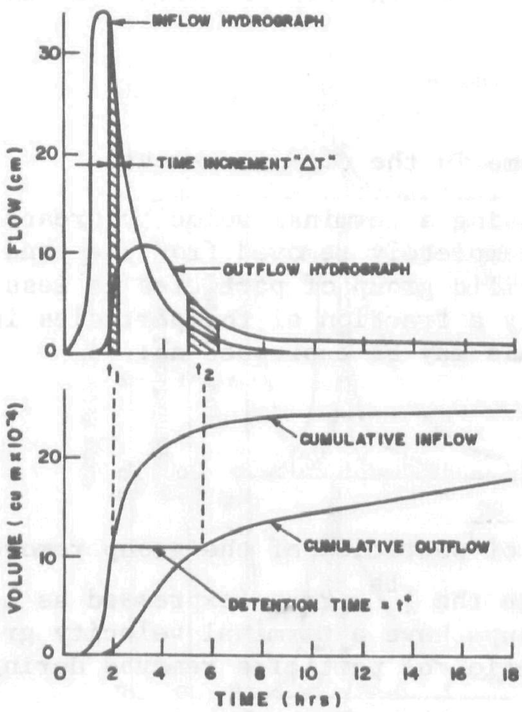

Figure 1. Method of establishing detention time.

PREDICTION OF PARTICLE REMOVALS

The settling behavior of suspended discrete non-flocculating in a laminar flow condition can be expressed by (after Camp, 1945):

$$v_t = \frac{g}{18\mu} (\rho_s - \rho_\ell) d^2 \tag{6}$$

where v_t is the terminal velocity, d = particle diameter, g = acceleration due to gravity, ρ_s = particle density, ρ_ℓ = fluid density and μ = fluid viscosity.

To utilize equation (6) in predicting the quantities of discrete particles removed in a pond it is necessary to assume that the particles are distributed randomly throughout the cross-section of the pond and that plug flow conditions exist.

For a particular time interval the average depth of water in the pond (H_{avg}) and the average outflow from the pond (0_{avg}) is known. It is then possible to calculate the critical settling velocity (v_c) for the interval. The critical settling velocity may be described as the velocity of the smallest particle which will be completely removed during the time increment. This velocity may also be defined as:

$$v_c = \frac{H_{avg}}{t^*} \tag{7}$$

where t* is the detention time of the time interval and is equal to:

$$t^* = \frac{S}{0_{avg}} \tag{8}$$

where S is the storage volume in the time increment.

Those particles possessing a terminal velocity greater than the critical settling velocity will be completely removed from the pond. If, however, the terminal velocity of a specific group of particles is less than the critical settling velocity, then only a fraction of the particles in the group will be removed. Mathematically this may be expressed as:

$$X_R = \frac{v_{t\,j}}{v_c} X_j \tag{9}$$

where X_R is the percentage of particles of the group removed and X_j is the total number of particles in the j^{th} group (expressed as a percentage of the total). If the first n groups have a terminal velocity greater than the critical velocity, the fraction of particles removed during the time increment can be given by:

$$X_{T\,i} = \sum_{j=1}^{n} X_j + \sum_{j=n+1}^{\infty} \frac{v_{t\,j}}{v_c} X_j \tag{10}$$

Using this equation in conjunction with the calculated concentration levels of the influent, as described in the previous section, the effluent concentration levels may be easily obtained.

To design the pond, it is necessary to decide which areas of the storm govern water quality. This decision is not obvious since maximum pollutant concentrations do not always coincide with the shortest detention time. For the case study, it was initially assumed that the concentration corresponding to peak flow conditions would be the controlling element of effluent water quality. Even if such an assumption was found to be invalid through simulations of the designs operation, such simulations would enable the quality controlling portions of the design storm to be identified. The detention time of this incremental volume of water corresponding to the peak of the inflow hydrograph is then estimated in the procedure outlined above.

Using SWMM (Huber et al, 1975), inflow hydrographs and pollutant loads were synthesized for storms corresponding to 5, 10 and 25 year return periods. Figure 2 summarizes the results.

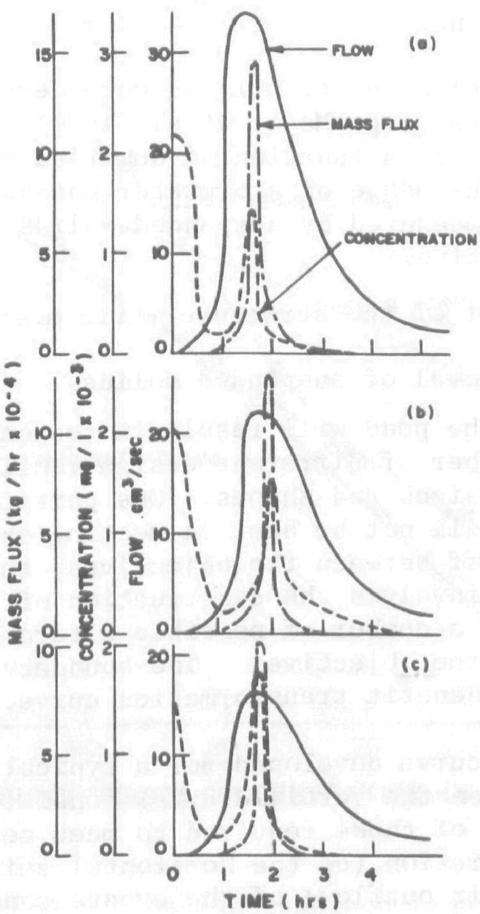

Figure 2. Relationship between flow, concentration and mass flux

MULTIOBJECTIVE ANALYSIS

Most water resource projects try to achieve maximum net benefits. This involves estimating all the costs and benefits for all feasible alternatives.

Project selection is then determined by identifying the project which provides the best return on the capital invested. The purpose is to identify optimal projects as a function of numerous relevant objectives. However, in many problems, many components of the costs are relatively easy to quantify in monetary terms. These include items such as construction costs, land costs and maintenance costs.

On the other hand, the benefits of increased water quality control are not only more difficult to identify, but are almost impossible to evaluate in monetary terms. These benefits include such aspects as increased property values downstream, higher values for aesthetic appreciation and an increase in biological life forms. In severe cases, pollutant removal may also decrease the cost of water treatment facilities downstream.

Although it is difficult to put a dollar value on these benefits, they do contribute significantly to the overall economic impact of the project and should not be ignored. The procedure used for decision-making should incorporate these values. One methodology useful for this purpose is multiple objective planning.

Multiple objective planning techniques were developed using economic production theory concepts (e.g. Major et al, 1970). Its purpose is to identify optimal projects as a function of numerous relevant objectives. Consider, for example, the usage of stormwater management ponds where water quality improvement, as measured by suspended solids removal, is at issue. There are two main objectives:

(i) to minimize the cost of the structure while meeting the post-development flow requirements,
(ii) to maximize the removal of suspended solids.

A particular design of the pond will result in responses with respect to both these objectives. A number of different design configurations exist consisting of different sizes and shapes. One particular design may best meet the objective 1 but it will not be best in meeting objective 2. To determine the nature of the tradeoff between the objectives, the first step of the multiobjective analysis involves the construction of a transformation curve. This involves evaluating a number of possible alternatives and plotting the points as a function of the objectives. The boundary of this technologically feasible set is the net benefit transformation curve.

The transformation curve developed for a typical case can be seen in Figure 3(a). The cost (on the vertical axis) consists of the construction and land costs in excess of those required to meet peak flow constraints. The suspended solids concentration (on the horizontal axis) is the highest concentration (in the weir outflow) of the events considered. Figure 3(b) shows the range of the transformation curve controlled by each storm.

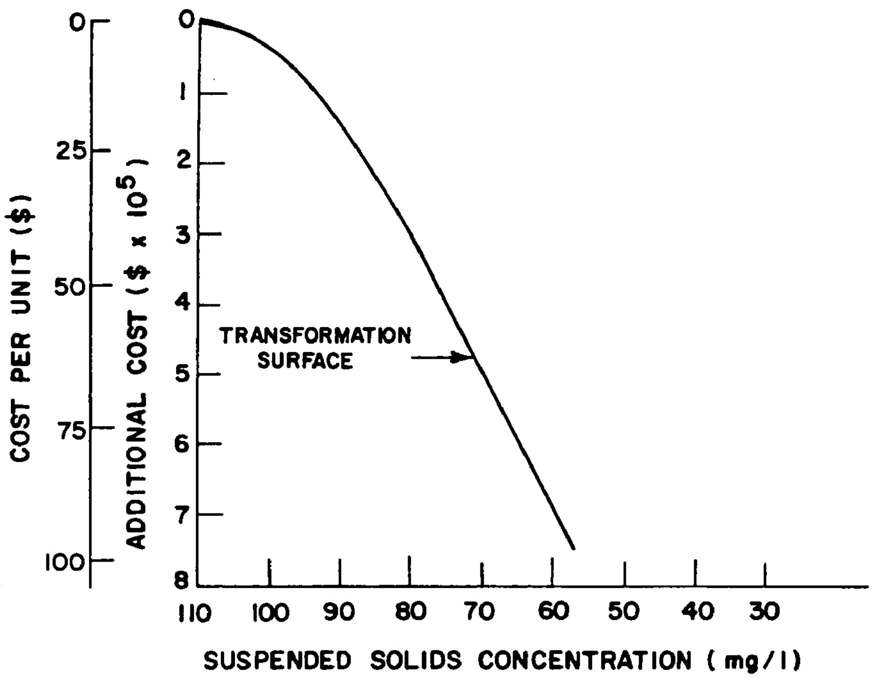

Figure 3(a). Transformation curve between additional costs and suspended solids concentration.

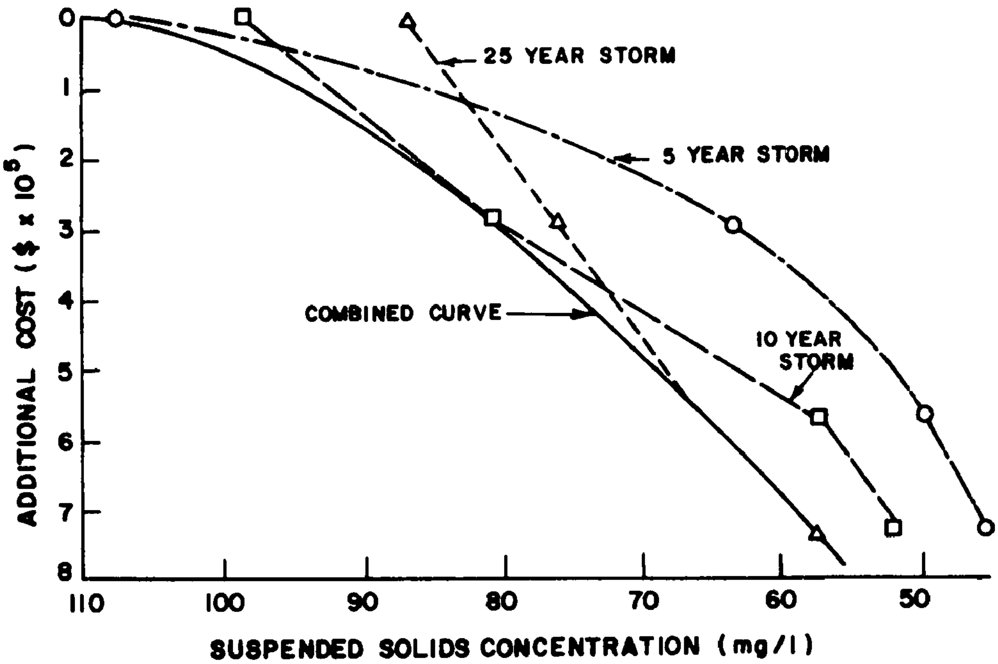

Figure 3(b). Elements creating combined transformation curve.

The next step in the use of multiple objective planning involves the development of preference functions. These functions represent the attitudes of the various decison-making bodies such as the developer, governmental authority, and the public. For a particular decision-maker, the decision-maker is equally content with any two points on the curve (see Figure 4).

The point of tangency between the transformation curve and the preference function indicates the trade-offs willing to be made by the decision-maker. Thus, together the transformation curve and the preference function indicates which design is desired by that group.

The construction of the transformation curves generally requires the interviewing of the various groups to obtain an indication of their perception of appropriate trade-offs between the objectives. The various curves illustrate the different attitudes of the various groups. The developer, for

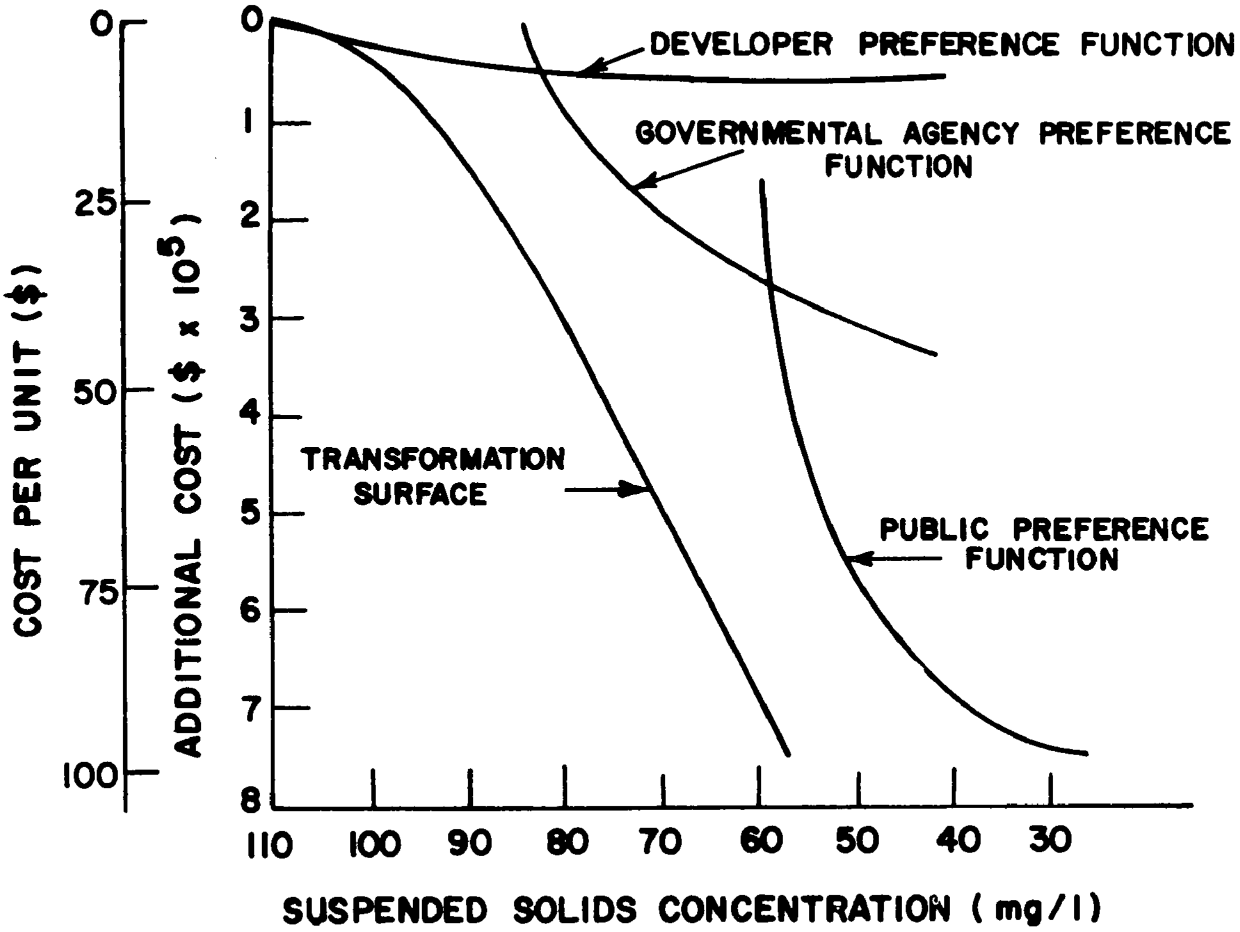

Figure 4. Illustration of multiobjective tradeoffs

example, is not willing to sacrifice much money for additional suspended solids removal and at some point he is unwilling to sacrifice any more money regardless of the level of suspended solids removal. This attitude is characterized by the horizontal nature of the preference function for low suspended solids concentrations and may be attributed to a minimum profit level. At some point, the additional money spent on water quality improvement will reduce the return on the developer's investment to such an extent that the overall development will represent a bad investment.

The public preference function usually places a much higher value on improved water quality. Usually such a curve has a maximum limiting value above which residents are unwilling to allow further degradation of the water body. The value could correspond to the concentration level at which severe ecological damage (e.g. fish kills) occurs. The other limiting case (that occurs when the public is unwilling to pay additional costs for water quality improvement) may correspond to a level when the pollutant or its effects are no longer visibly noticeable.

The preference function of the governmental authority may usually be found somewhere between that of the public and that of the developer. If governmental guidelines concerning the quality of urban runoff exist, then this curve will also possess an upper bound for the quality of urban runoff. The presence of an upper bound on the money spent to achieve improved water quality may or may not exist depending on current governmental policies.

CONCLUSIONS

Stormwater management ponds have received widespread implementation in Canada. With increasing frequency, however, the criteria used in the ponds are going beyond flood-related concerns to encompass many other uses.

Multiple objective planning techniques are useful to identify optimal (and near optimal) projects as a function of numerous relevant objectives. These procedures have utility to improve the understanding of how different the various decision-makers are in their perception of the best design configuration.

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DETERMINATION OF RUNOFF CHARACTERISTICS OF FLATWOODS WATERSHEDS[1]

K. L. Campbell, J. C. Capece and L. B. Baldwin[2]
Agricultural Engineering Department
Institute of Food and Agricultural Sciences
University of Florida
Gainesville, Florida 32611

ABSTRACT

Several methods of estimating stormwater runoff total volume and peak discharge are evaluated as to their performance on watersheds of Florida's Flatwoods Resource Area. Characteristics of these watersheds include extremely flat relief, sandy soils, dynamic water tables, and scattered wetlands. Data collected by the U. S. Geological Survey and South Florida Water Management District from five small (20-3600 acres), agricultural watersheds (improved and unimproved pasture) served as the basis of evaluation. All total volume estimation techniques examined rely upon the SCS runoff equation. Best results were achieved with methods which included antecedent depth to the water table as a measure of watershed storage potential. Runoff peak rate estimation techniques ranged in approach from empirical formulas to an overland flow simulation model. For the original methods examined, standard errors of estimate were inversely proportional to model sophistication. Two peak rate estimation methods, the CREAMS hydrologic model equation and the SCS unit hydrograph method, were modified to better reflect observed data.

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2 The authors are, respectively: Associate Professor, Instructor and Associate Professor, Agricultural Engineering Department, University of Florida, Gainesville, Florida 32611.

INTRODUCTION

Many techniques have been developed to estimate stormwater total runoff volume and peak discharge rates from small watersheds. However problems arise when these methods are applied to the unusual hydrologic conditions found in Florida's Flatwoods Resource Area. Watersheds of this area typically have very flat slopes, extremely permeable sandy soils, high water tables, and wetlands scattered throughout their basins. Such characteristics are unlike those of the watersheds which served as the models for the development of most runoff prediction methods. The problems introduced by these atypical watershed conditions are often compounded when the methods are called upon to predict runoff resulting from rainfall events for which they were not intended i.e., frequent, instead of extreme (design), events.

Studies which document the accuracy of standard runoff prediction techniques as applied to Florida's flatwoods watersheds under a range of rainfall events are not currently available. Hydrologists, engineers, and water resource managers are therefore forced to make decisions based upon runoff estimates resulting from methods which, although generally accepted, are not necessarily accurate under these particular watershed conditions. The users often appreciate the errors and limitations associated with their runoff estimates, but do not have sufficient information with which to offer improvements. The research described in this paper represents an effort to help fill the existing information gap.

Data collection sites for this study are within the Lower Kissimmee River and Taylor Creek-Nubbin Slough Basins north of Lake Okeechobee. The predominant soil associations for both basins are Myakka-Immokalee-Waveland and Wabasso-Felda-Pompano (1). Natural vegetation consists primarily of wet and dry prairie grasslands and pine-palmetto forests. In the depressional areas, wetlands species predominate. Land use in the two basins is dominated by improved and unimproved pasture, claiming about 75% of the total area by 1980 (2, 3).

The means of transformation from a natural marsh and slough system to agricultural use has been drainage improvement achieved through ditching. Extensive channel networks combined with extremely low watershed slopes (<0.5%) make delineation of watershed boundaries a difficult task in some cases. Drainage patterns can, in fact, shift depending upon rainfall patterns and runoff magnitude.

Hydrologic data from five watersheds ranging in size from 20 to 3600 acres and located within the Lower Kissimmee River and Taylor Creek-Nubbin Slough Basins were collected between 1979 and 1983 in conjunction with the Upland Detention/Retention Demonstration Project. More detailed site and data descriptions can be found in the project report (4).

METHODS

TOTAL RUNOFF VOLUME

The SCS runoff equation serves as the basis for the total volume estimation methods examined in this report and analyzed by Konyha et al. (5) and can be written as:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad [1]$$

where P = 24-hour rainfall depth and
S = watershed storage parameter.

Each of the following methods uses a distinct technique for arriving at the storage parameter, S. These methods are referred to in this paper as NEH-4 (6), SCS-Florida (7), DRM (8), ARS (4, 9), CR-1 (5, 10), CR-2 (5,11), and CR-WT (12). NEH-4 is the standard SCS curve number method as described in the SCS National Engineering Handbook-Section 4, Hydrology (6). SCS-Florida is a revision of this method from the SCS Florida Interim Procedure report (7) which uses antecedent moisture condition II for all events. DRM is a method described in the South Florida Water Management District (SFWMD) Regulatory Manual (8) which makes the storage parameter, S, in equation 1 a direct function of the depth to water table. ARS is a method developed by modifying the available storage relationship in DRM based on research data from Speir et al. (9). CR-1 uses an algorithm from the CREAMS model (10) to adjust the watershed storage parameter as determined by the ARS method to allow for influences of management practices on runoff volume. CR-2 is similar to CR-1 except that it uses a simple soil moisture accounting model to estimate the storage parameter instead of relying on water table data. CR-WT uses the full hydrology component of the CREAMS model in a modified form to account for a fluctuating shallow water table by preventing deep percolation out of the soil profile (12). Each of these methods is described in more detail in the project report (4).

STORM RUNOFF PEAK RATE

A variety of approaches are available for the routing of stormwater to arrive at peak discharge rates. Several techniques representing a range of complexity levels have been applied to the Florida flatwoods. The following methods were evaluated, beginning with the very empirical and progressing through to the more theoretical approaches:

1. Cypress Creek Formula (11),
2. CREAMS peak rate equation (10, 13),
3. SCS Chart Method (7, 14),
4. SCS Unit Hydrograph Method (6, 15), and

5. SFWMD Model (16, 17) used to generate graphs in District Regulatory Manual IV (8).

Each of these methods is described by Capece et al. in the project report (4). Two methods, the CREAMS equation and the SCS Unit Hydrograph method, were modified in an effort to improve their performance. The results are summarized in the following section of this paper and the procedures used in modifying the methods can be found in the project report (4).

RESULTS

STORM RUNOFF TOTAL VOLUME

Seven techniques for estimating stormwater runoff volume were applied and evaluated on an event basis. The selected storms measured 0.70 or more inches of rainfall in 24 hours and may or may not have produced measurable runoff. The seven runoff volume estimation methods applied to the data set were: NEH-4, SCS-Florida, DRM, ARS, CR-1, CR-2, and CR-WT as described earlier.

Tables 1 and 2 present the same results as standard error of estimates determined with the following equation:

$$\varepsilon = \left[\frac{\sum_{i=1}^{n} (Q_i' - Q_i)^2}{n - 1}\right]^{0.5} \qquad [2]$$

where ε = standard error of estimate in inches,

Q_i' = predicted runoff volume for event i in inches,

Q_i = measured runoff volume for event i in inches, and

n = total number of storm events.

Each table represents technique performance as applied to selected classes of events: all daily rainfall events equal to or exceeding 0.70 inches, and a subset which produced runoff equal to or exceeding 0.50 inches.

Standard errors corresponding to "all" sites do not weight each watershed equally. Instead, the overall method standard error of estimate is most heavily weighted toward the sites which had more usable events. The ranking

TABLE 1. STANDARD ERRORS OF RUNOFF VOLUME ESTIMATES, IN INCHES, FOR ALL EVENTS

Site	Method						
	NEH-4	SCS-FL	DRM	ARS	CR-1	CR-2	CR-WT
Armstrong	0.57	0.52	0.40	0.38	0.44	0.70	0.52
Peavine	0.66	0.57	0.45	0.45	0.45	0.70	0.61
SEZ Dairy	0.31	0.57	0.52	0.40	0.46	0.61	0.30
Bass West	1.12	0.86	0.50	0.46	0.53	0.64	1.11
Bass East	0.64	0.54	0.57	0.55	0.57	0.69	0.84
All	0.73	0.63	0.47	0.44	0.48	0.66	0.74
Site Ranking	(6)	(4)	(2)	(1)	(3)	(7)	(5)

TABLE 2. STANDARD ERRORS OF RUNOFF VOLUME ESTIMATES, IN INCHES, FOR EVENTS WITH MEASURED RUNOFF EQUAL TO OR EXCEEDING 0.50 INCHES

Site	Method						
	NEH-4	SCS-FL	DRM	ARS	CR-1	CR-2	CR-WT
Armstrong	1.38	0.79	0.81	0.54	0.93	0.75	1.12
Peavine	1.09	0.58	0.65	0.59	0.65	1.07	1.05
SEZ Dairy	0.71	0.64	0.71	0.47	0.76	0.60	0.72
Bass West	1.85	1.37	0.78	0.72	0.83	0.88	1.62
Bass East	1.31	0.91	1.01	1.04	1.08	1.06	1.53
All	1.36	0.93	0.76	0.67	0.79	0.90	1.32
Site Ranking	(7)	(2)	(5)	(1)	(4)	(3)	(6)

corresponding to "all" sites was determined by comparing the sum of the methods' performance ranking for each site and, therefore, weights performance on each watershed equally.

Generalizations can be drawn from these rankings regarding technique overall performance and trends through changing runoff volume. The ARS method consistently performed better than all other methods. The SCS-Florida method demonstrated improved accuracy as runoff volume increased, as would be expected of a method intended for design applications. The CR-1 method performed very well on the smaller events, but not as well on the larger events. The DRM method gave results very similar to ARS and CR-1 for small events and also demonstrated decreased accuracy when applied to the larger runoff events. The CR-2 method performed poorly on the small events, but improved somewhat on larger events. Both the NEH-4 and CR-WT methods produced consistently inaccurate estimations of runoff volume.

STORM RUNOFF PEAK RATE

The following sections present results of the peak rate estimation techniques as applied to each watershed and all runoff events. Performance is quantified as standard error of estimate, in percent, and average error of estimate, in percent:

$$\varepsilon = \left[\frac{\sum_{i=1}^{n}\left(\frac{q_i' - q_i}{q_i}\right)^2}{n-1}\right]^{0.5} \qquad [3]$$

$$\xi = \left[\frac{\sum_{i=1}^{n}\left(\frac{q_i' - q_i}{q_i}\right)}{n}\right] \qquad [4]$$

where ε = standard error of estimate in percent,

ξ = average error of estimate in percent,

q_i' = predicted peak rate for event i in cfs,

q_i = measured peak rate for event i in cfs,

n = total number of runoff events.

The average error for each site describes a method's tendency to overpredict or underpredict while the standard error quantifies error absolute magnitude. Rainfall events greater than 0.70 inches and having measurable runoff were included in the data base for this analysis. Results described as applying to "all" sites are biased toward the sites having more usable runoff events. Standard errors of estimate for each peak rate method are summarized in Table 3.

TABLE 3. RESULTS SUMMARY FOR PEAK RATE ESTIMATION TECHNIQUES AS APPLIED TO EVENTS WITH MEASURED RUNOFF EQUAL TO OR EXCEEDING 0.50 INCHES

Site	Peak Rate Estimation Technique						
	Cypress	CREAMS	SCS-Chart	SCS-UH	SFWMD	CR-Mod	UH-Mod
Armstrong	256.*	1002.	46.	106.	67.	21.	21.
Peavine	946.	2770.	142.	430.	55.	33.	77.
SEZ Dairy	656.	1764.	117.	192.	64.	34.	51.
Bass West	363.	2069.	201.	188.	29.	21.	18.
Bass East	1050.	7166.	1075.	44.	479.	86.	26.
All	715.	3511.	461.	254.	181.	42.	45.

* Results are reported as standard error of estimate, in percent.

Runoff events measuring less than 0.50 inches tended to produce erratic results. Because estimation errors are expressed as a percent of measured peak, very small events are prone to produce large errors of estimate. Another problem was that with small quantities of measured runoff, ground water discharge becomes more significant and produces atypical hydrographs. For these reasons, emphasis is placed on peak rates predicted for runoff events equal to or exceeding 0.50 inches.

Cypress Creek Formula

Predictions from the Cypress Creek Formula are compared against measured peaks in Table 3. As previously described, prediction errors associated with the smaller events were larger than the error reflected in Table 3.

The standard and average errors were comparable in magnitude and average errors are all positive. Thus, the method consistently resulted in large overpredictions of measured peak discharge. Standard errors ranged from 200% for Armstrong (the largest watershed) to 1000% for Bass East (the smallest watershed). Even when the effect of transforming a 24-hour maximum rate into an instantaneous rate was removed, the method still overpredicted.

CREAMS Equation

The standard CREAMS equation performed worse than any other method examined in this study. It consistently overpredicted by an order of magnitude or more (see Table 3). The results when examined graphically indicated that the estimation error was fairly consistent for all sites.

A regression of the CREAMS model formulation against measured data yielded this modified version of the equation:

$$q_p = 4.52(DA^{1.06})(CS^{0.77})(LW^{0.389})(Q^{0.87(DA^{-0.20})}) \quad [5]$$

where q_p = peak runoff rate in cfs,

DA = drainage area in mi^2,

CS = main channel slope in ft/mi,

LW = watershed length to width ratio, and

Q = daily runoff volume in inches.

When reapplied to the data base, performance of this equation was good. CR-Mod in Table 3 does not represent an independent evaluation of the modified CREAMS equation, but simply reflects the regression fit to the data. This equation resulted in an R^2 of 0.96. The standard error of estimate associated with equation 5 ranged from 20% for Bass West to 85% for Bass East.

SCS Chart Method

Performance comparisons for the SCS Chart Method are presented in Table 3. Unlike the Cypress Creek Formula, large events produced results similar to those for all events. This method also tended to overpredict peak discharge. Maximum and minimum standard errors of estimate were 1000% and 50%, corresponding to Bass East and Armstrong Slough, respectively.

SCS Unit Hydrograph Method

Evaluation of the unit hydrograph method was conducted in three steps: (1), evaluation of standard SCS methodology; (2), evaluation and modification of certain aspects of the method; and (3), re-evaluation of the unit hydrograph method implementing various modifications.

Based on the triangular unit hydrograph, the standard estimate for K' (484) describes a hydrograph whose recession is 1.67 times as long as its time to peak. Mockus (6) notes that this K' value has been known to vary from 600 in steep terrain to 300 in flat swampy country. For the Delmarva peninsula, which includes Delaware and parts of Maryland, Welle et al. (18) concluded that a value of 256 is more appropriate. The watersheds they examined were small with sandy soils and slopes in the range of 2%. The U.S. Army Corps of Engineers (19) studied records from several large watersheds in Central and South Florida (the entire Kissimmee River Basin being one) and determined an appropriate time factor for use in a similar peak discharge equation. Miller and Einhouse (20) translated this factor into the SCS form, arriving at a value of 284 for K'.

Table 4 presents results from application of the standard K' factors (300 and 484). SFWMD assumed rainfall distributions were used for all events. Best peak estimates were obtained using a K' of 300. However this method still tended to overpredict discharge peaks by about 200%.

TABLE 4. RESULTS SUMMARY FOR SCS UNIT HYDROGRAPH METHOD

Site	K' = 300		K' = 484	
	Std. Error	Avg. Error	Std. Error	Avg. Error
Armstrong	106.*	88.	230.	200.
Peavine	430.	380.	724.	646.
SEZ Dairy	192.	142.	352.	272.
Bass West	188.	175.	337.	316.
Bass East	44.	28.	112.	94.
All	254.	195.	439.	354.

* Errors are reported in percent.

The second step of the evaluation was to determine best-fit K' factors using various estimates of time to peak, both measured and assumed rainfall time-distributions, and different classes of runoff events. The SFWMD assumed rainfall distribution yielded almost identical results to those derived from measured distributions. Differences in best-fit K' values were apparent when comparing the two runoff event classes. Factors for events less than 0.50 inches were 20-30% higher than factors for the larger class of events. The scatter among K' values for each site is also greater for smaller events. Focusing in on the K' solved using assumed rainfall distributions and events greater than 0.50 inches, almost all were computed to be less than 100.

Table 5 shows the site variability of best-fit factors. Here the average K' is calculated over n events for each site and s is the standard deviation associated with that average. The trend among sites was for an increasing K' value with decreasing watershed percent wetlands. Results reported for Peavine Pasture differentiate between the large and small drainage area conditions. The smaller factor associated with the 1800 acre watershed may be due to channel block effects.

Also included in Table 5 are the best-fit factors for the large runoff events associated with Hurricane David. Runoff from this event ranged between 2.5 and 5.2 inches, depending upon the specific site. Armstrong data for this period are documented as being "estimated." Questions also exist regarding the actual contributing area for this event as well as other events at other sites. Examination of the SCS unit hydrograph equation shows that when measured runoff data are used, the influence of errors in drainage area estimates upon peak rate calculations is confined to the T_p term. The multiplication of the depth and area terms in the numerator of the equation yields volume. This is the inverse of the calculation used to estimate runoff depth (measured volume divided by estimated contributing area) and thus negates the influence of drainage area estimates.

TABLE 5. RESULTS SUMMARY FOR SCS UNIT HYDROGRAPH K' FACTOR OPTIMIZATION USING SFWMD ASSUMED RAINFALL DISTRIBUTION, RUNOFF EVENTS EQUAL TO OR EXCEEDING 0.50 INCHES, AND MODIFIED-FIXED LAG ESTIMATES

Site	n Events	Maximum*	Minimum†	$\bar{K}$‡	s	David§
Armstrong	5	119.	62.	83.	22.	88.
Peavine (1800)	10	74.	27.	50.	15.	66.
Peavine (775)	3	76.	53.	66.	12.	-
SEZ Dairy	4	101.	39.	70.	26.	-
Bass West	16	107.	59.	88.	12.	118.
Bass East	7	84.	66.	72.	8.	70.
Average				72.	13.	

* Maximum observed optimized event K' factor.

† Minimum observed optimized event K' factor.

‡ $\bar{K}$ values represent an average of all events for a site.

§ Based on available data for rainfall associated with Hurricane David (9-3-79).

Incorporating results from the K' analysis, the incremental unit hydrograph method was re-applied to the data base. Like the initial evaluation, only SFWMD assumed rainfall distributions and runoff events greater than 0.50 inches were examined. Best results shown in Table 3 under UH-Mod were achieved with a K' of 75.

SFWMD Model

The SFWMD overland flow computer model was applied to runoff events exceeding 0.50 inches. Results are summarized in Table 6. The model underpredicted on most sites. However, where it did overpredict, the percent error was high (Bass East). Best results were associated with the Bass West and Peavine watersheds. The large Peavine Pasture watershed is believed to respond in a sheetflow manner and should be described well by an overland flow model. For the large watersheds with significant channel effects (Armstrong

Slough and SEZ Dairy), the model underpredicted as would be expected. However for the smallest watershed (Bass East), where the overland flow approximation would appear most applicable, results were not good. The observed overprediction is probably due to the length-to-width ratio of the pasture. It is wide, about 1700 feet, and only 500 feet long. This shape would simulate as a very high peak-producing watershed.

TABLE 6. RESULTS SUMMARY FROM SFWMD OVERLAND FLOW COMPUTER MODEL AS APPLIED TO RUNOFF EVENTS WITH MEASURED RUNOFF EQUAL TO OR EXCEEDING 0.50 INCHES

Site	Standard Error	Average Error
Armstrong	67.*	-60.
Peavine (1800)	59.	10.
Peavine (775)	50.	-35.
SEZ Dairy	64.	-46.
Bass West	29.	-8.
Bass East	479.	410.
All	181.	48.

* Errors are reported in percent.

Summary

When compared with one another (Table 3), the methods demonstrate magnitudes of error inversely proportional to their degree of complexity. With decreasing overall standard error of estimate, the original methods line up as: CREAMS, 3500%; Cypress Creek Formula, 700%; SCS Chart, 400%, SCS unit hydrograph, 250%; SFWMD, 180%. The CREAMS and Cypress Creek Formula should be reversed based upon complexity level, however the CREAMS equation was not developed using Florida data, while the Cypress Creek Formula is described as being applicable to the Florida flatwoods.

Fitting the CREAMS equation and SCS unit hydrograph approach significantly improved the performance of both methods. Each achieved between 40% and 45% standard error of estimate with little bias toward over- or under-predictions.

SUMMARY AND CONLUSIONS

TOTAL VOLUME EVALUATION

The SCS runoff equation was developed for application to large design events occurring on small watersheds. However, in many instances it has been applied to smaller rainfall events with little or no consideration given to accuracy implications. Specific techniques employed to determine the watershed storage parameter, an input to the SCS runoff equation, have not been sufficiently evaluated as to their suitability for atypical watershed conditions.

Evaluations of the SCS equation and specific methods for determining its inputs demonstrate that large errors can be associated with runoff estimates for smaller events. For the seven methods examined, overall standard error of estimates ranged from several hundred to fifty percent. For both the larger and smaller events, best estimates of runoff volume resulted from techniques which incorporated antecedent water table conditions.

Three of the methods (DRM, ARS, and CR-1) relied upon measured water table elevations and performed similarly on small events. The ARS method consistently performed best on all event classes. The CR-1 method incorporates the ARS storage relationship, but has the added advantage of accounting for factors other than water table depth via the SCS curve number. This offers latitude useful in evaluating changes in runoff volume resulting from alternative land use patterns and agricultural practices. The CR-2 method has the same advantages, but rather than water table history or assumptions, a rainfall history is required. This method did not perform as well as CR-1. The SCS-Florida method considers strictly land use and soils, ignoring variations in watershed wetness. Therefore its use could lead to significant runoff estimation errors when applied to large events falling upon saturated watersheds. Neither the NEH-4 nor CR-WT methods should be used for runoff estimation on an event basis.

Estimates of runoff volume and the evaluation of prediction methods are more sensitive to errors in data collection and drainage area determination than are peak rates. However results demonstrate that techniques which incorporate water table levels (total available soil storage) can be expected to yield more accurate estimates of runoff volume for flatwoods watersheds.

PEAK RATE EVALUATION

Results of this study demonstrate that more accurate estimates of runoff peak rates can be expected as models progress from the empirical to the more physically based. However when empirical models are tailored to specific watershed conditions, results may be comparable to those from more complex models. As watershed conditions change or changes are anticipated, physically based models again become more reliable than empirical techniques.

The two extremes of empirical and physical models are represented by the CREAMS equation and the SFWMD overland flow computer program. The overland flow model performed best of all the original methods examined, however, it still demonstrated considerable overall error.

With modifications, estimation error was significantly reduced in the CREAMS equation and SCS unit hydrograph method. For the CREAMS equation, overall standard error of estimate was reduced from 3500% to 42%. For the unit hydrograph method, modifications reduced the overall standard error estimate from 250% to 45%. Between the two modified methods, the SCS method is more versatile and should be more transportable to other flatwoods watersheds. The SCS technique is capable of handling multiple-day (complex) events, whereas the CREAMS equation does not allow superposition.

Significant unit hydrograph results indicate that the SCS recommended triangular hydrograph factor, 300, is too high. Analyses indicate that a value less than 100 is more appropriate for Florida's flatwoods watersheds. Also noteworthy were the almost identical peak rate estimates derived from measured and assumed rainfall time-distributions. Discharge hydrographs from flatwoods watersheds are much more attenuated and produce much lower peaks than most other small watersheds of the United States.

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STORMWATER MANAGEMENT MODEL APPLICATION TO A PEATLANDS REGION IN COASTAL NORTH CAROLINA

by: Robert E. Dickinson
Warren Pandorf
Larry J. Danek
Environmental Science and Engineering, Inc.
Gainesville, Florida 32602

ABSTRACT

The runoff from a North Carolina peatlands region, an area with a high drainage density, was simulated using a modified version of the Storm Water Management Model (SWMM) RUNOFF block. The RUNOFF block was changed to simulate two types of pervious flow: (1) surface and overland flow from the peat areas adjacent to the drainage ditches, and (2) slower subsurface flow from the inter-ditch peat areas. In general, the peat areas near the canals and ditches contribute to the peak portion of the runoff hydrograph while the inter-ditch areas contribute to the recession curve of the runoff hydrograph. The model was calibrated to 28 rainfall-runoff events measured at two small peat sites: (1) a 10-acre shrubland (gallberry) site and (2) a 10-acre harvest (bare soil) site.

INTRODUCTION

The Pamlico-Albemarle peninsula of North Carolina has extensive deposits of peat soil along with freshwater marshes, swamps, and upland hardwood, pine, and mixed forests. The majority of the peninsula was once covered with dense verdant vegetation that impeded stormwater runoff to the North Carolina estuaries (1). The peninsula was first developed for farming in 1787, with the completion of a canal that drained 10,000 acres of peatland (2). This development process accelerated during the period from 1930 to 1950 with the construction of numerous main canals, cross-connector canals, and drainage ditches. This canal system has a high drainage density (nearly 20 miles of canals and ditches per square mile of land area) which increases the peak storm flows and the sediment load to the estuaries of North Carolina.

Ironically, much of the ditched pocosin area was found to be unsuitable for farming. The ditched peat areas were either converted to pasture, left fallow, or never developed. During the 1980's, a plan was developed to harvest 15,000 acres of the ditched peat area for conversion to methanol, with the methanol being used as a fuel additive (3). As part of the permitting process for the peat-to-methanol plant, an environmental monitoring network for water quality and quantity was set up at the project site (see Figure 1 for a location map of the project area).

SWMM Version III (4) was chosen to simulate the runoff from the project site for the following reasons:

1. Its output was compatible with the input requirements of RECEIV (5), which was being used to simulate water quality in the receiving water estuary;
2. It was capable of both single-event and continuous rainfall simulation; and
3. It could simulate the complex canal and lake system to the extent of detail required by permitting agencies.

The problem with using the model, however, was that the overland flow portion of the SWMM RUNOFF block was physically unrepresentative of the runoff process from the peat area. The runoff from a peat area is primarily subsurface or interflow. The runoff-generating mechanism of RUNOFF was modified to successfully simulate stormwater runoff from ditched peat areas. The remainder of this paper describes the modified SWMM application to two small (10-acre) sites in the project area. The two sites, one covered by gallberry shrub (Site C on Figure 2) and the other a bare soil peat area (Site H on Figure 2), are 165 feet wide and 2,640 feet long. The sites are bisected by a 4-foot-wide drainage ditch which flows to a cross-connector canal and then to a main drainage canal. The land area is mildly sloped (1-percent slope) to the drainage ditches and the ditches have a slope of 0.1 percent. Rainfall and runoff data for 28 storm events were gathered between February 1983 and July 1983 and were used to calibrate the runoff model.

KINEMATIC WAVE STORAGE MODEL

A peat area can be described as a perched water table in which there is little hydrologic interaction with the deep aquifer because of the impermeable soil underlying the peat (1). The only mechanisms for the removal of rainfall are evapotranspiration and runoff.

The peat runoff mechanism is influenced by the presence of the bisecting drainage ditches. When the water table is below the bottom of the ditch, there will be no runoff except for a small amount due to direct rainfall in the near ditch area. During the seasons of the year in which the water table is above the bottom of the drainage ditch, the runoff into the ditch will come from the saturated peat areas adjacent to the drainage ditch. The peat is saturated due to a rising water table fed by infiltration from above and by laterally inflowing baseflow (6).

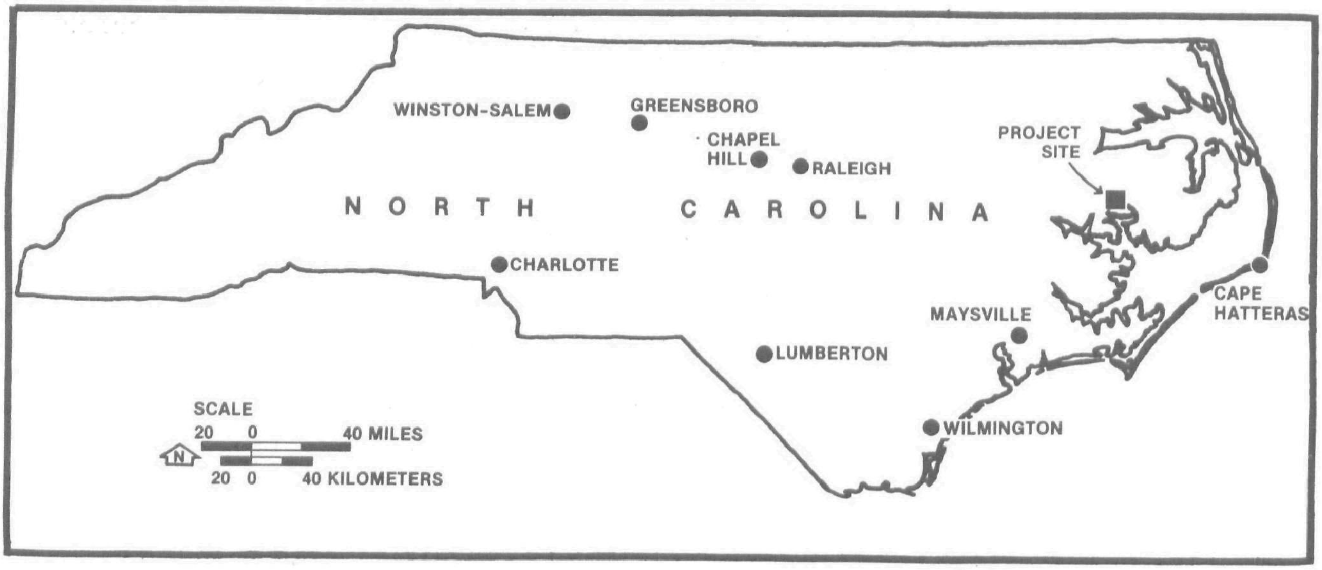

Figure 1. Location map.

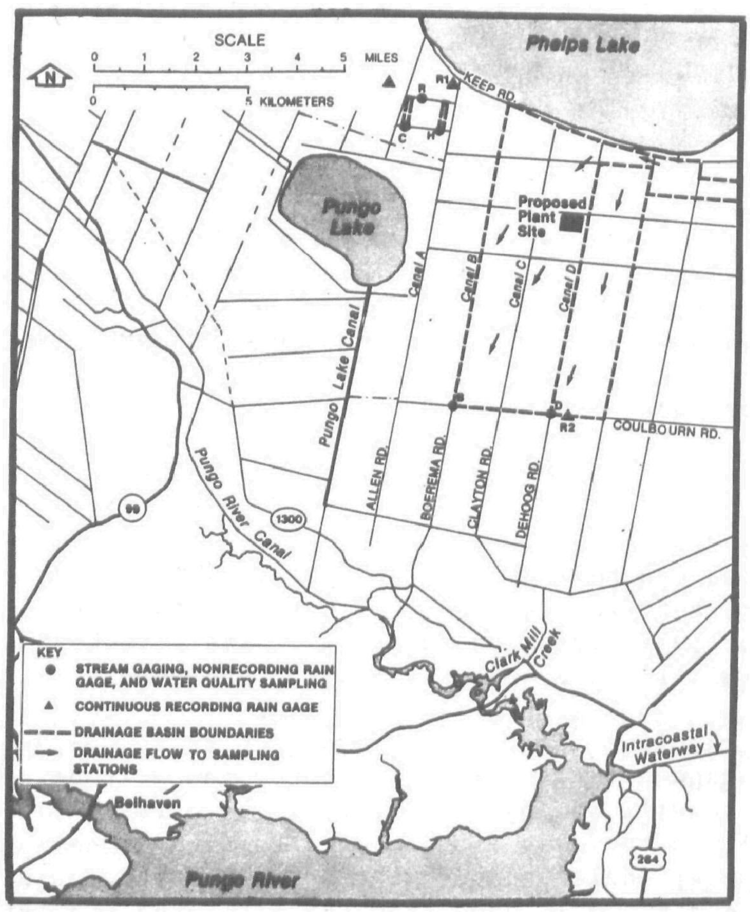

Figure 2. Storm event sampling locations — February-July 1983.

The existing SWMM runoff-generating mechanism, which uses a version of the kinematic wave equation with Manning's velocity equation, is best suited for turbulent overland flow (4, 7). However, subsurface flow models using alternative kinematic wave equations for a runoff mechanism were found to be appropriate for soils with large macropores (8, 9, 10). The peat area, which has large macropores due to undecayed tree trunks, branches, and twigs, is modeled using a kinematic wave storage model.

The kinematic wave equation relates depth of water in storage to outflow. Kinematic wave equations are applicable in situations in which all the terms of the mass continuity equation are negligible compared to bottom slope and friction. The equation has the general form of:

(1) $$q = \alpha d^m$$

where: q = flow,
α = coefficient,
d = depth of water in peat, and
m = exponent.

The coefficient α and the exponent m vary according to flow regime, i.e. laminar or turbulent flow. The exponent m has a value of 1.5 using the Chezy turbulent flow equation, a value of 1.66 using Manning's turbulent flow equation, and a value of 3.0 for laminar flow (7). Horton in 1983 found an average value of $m = 2.0$ for a mixture of laminar and turbulent flow (11). The calibrated exponent for runoff from the two peat areas was 2.66. Two α values were calibrated per drainage area: (1) the first α is the coefficient for the subsurface and overland flow from the peat areas adjacent to the drainage ditch (α_1), and (2) the second α is the coefficient for the slower subsurface flow from the inter-ditch area (α_2). The α_1 and α_2 coefficients incorporate the effect of slope, roughness, and undefined factors of influence in the flow model.

The drainage sites were modeled with two contributing runoff source areas. The peat area nearest the drainage ditch contributes to the peak portion of the storm hydrograph (first source area), while the inter-ditch area contributes to the recession curve portion of the storm hydrograph (second source area). There is no interconnection between the two areas; however, there is a direct model connection between each source area and the outlet of the drainage ditch.

The first source area was assumed to always have an initial depth of zero. The second source area has an initial water depth corresponding to the base flow immediately preceding the calibrated storm event. The initial depth is calculated as:

(2) $$d_o = (Q_o/\alpha_2)^{3/8}$$

where: d_o = initial depth of water in peat (feet),
α_2 = kinematic wave coefficient, and
Q_o = base flow (cfs).

An initial depth of zero corresponds to a water table elevation equal to the elevation of the bottom of the drainage ditch.

The vertical boundaries of the model extend from the surface to the bottom of the drainage ditch. The model is constrained to never have a negative depth. A water balance is maintained, i.e. runoff plus evaporation and storage equals rainfall.

RESULTS AND DISCUSSION

A visual comparison of the measured runoff at the shrubland site (Figure 3) versus the measured runoff at the harvest site (Figure 4) shows that the harvest site had larger peak flows. Because of an extensive root mat, the shrubland site attenuates the peak discharge and lengthens the runoff time compared to the bare soil harvest site.

The predicted versus measured calibration results for 13 storm events from the shrubland site are presented in Table 1, and the results for 15 storm events from the harvest site in Table 2. Presentations of the comparison of measured versus predicted runoff for selected events at the control site and harvest site are shown in Figures 5 and 6, respectively.

Each drainage area had a rain gage located next to the V-notch weir but due to the shape of each drainage area (2,640 feet long and 165 feet wide), a significant portion of the runoff error may be the result of rainfall variability during localized storms. For example, the rainfall for Storms 7, 8, and 9 was observed to be quite variable among the onsite rain gages. If this rainfall was underestimated from the shrubland site, this may explain the high runoff rate (90.6 percent) measured for these three storms.

The model closely predicted runoff from both sites for three seasons of storm event data (winter, spring, and summer). The shapes of the hydrographs, peak flows, and total volumes were matched to the field data by adjusting the coefficients of the kinematic wave storage model.

The result was a slightly more complicated runoff model that was more representative of the actual physical conditions than the original SWMM runoff mechanism.

The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

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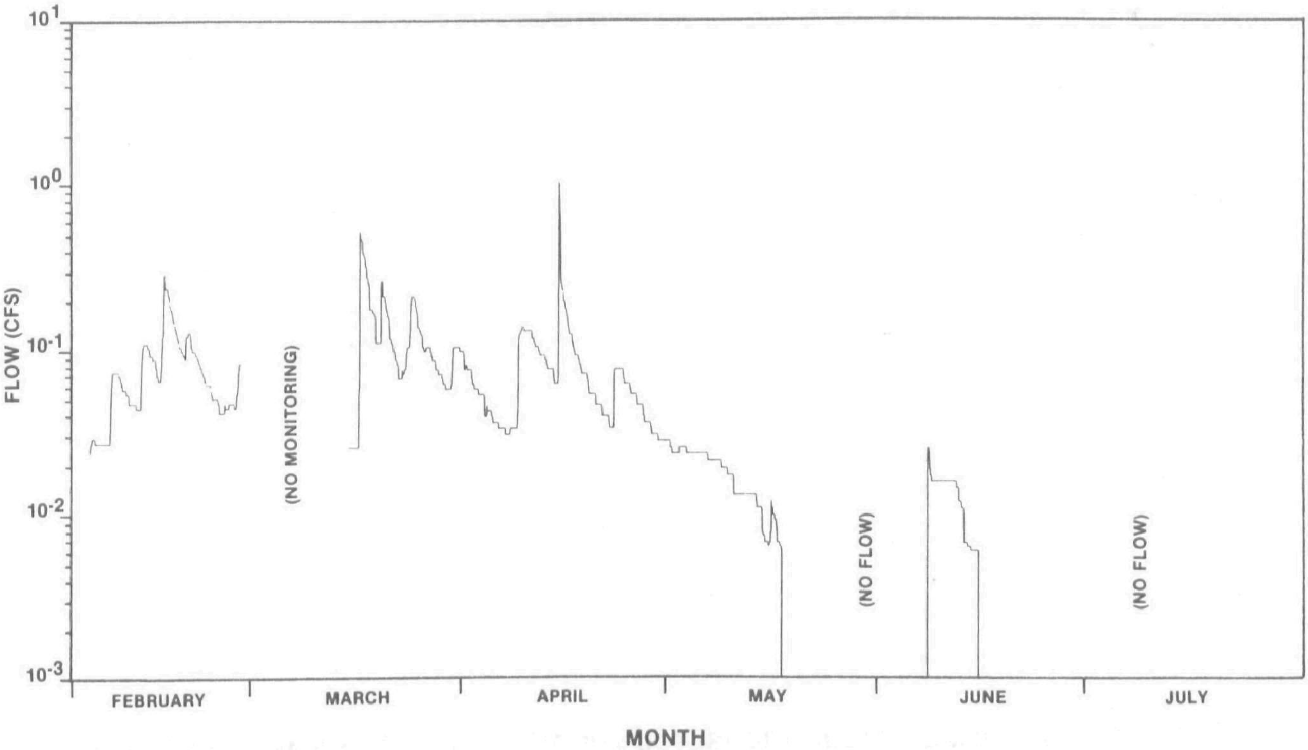

Figure 3. Flow hydrograph at Station C (Control Site) — February-July 1983.

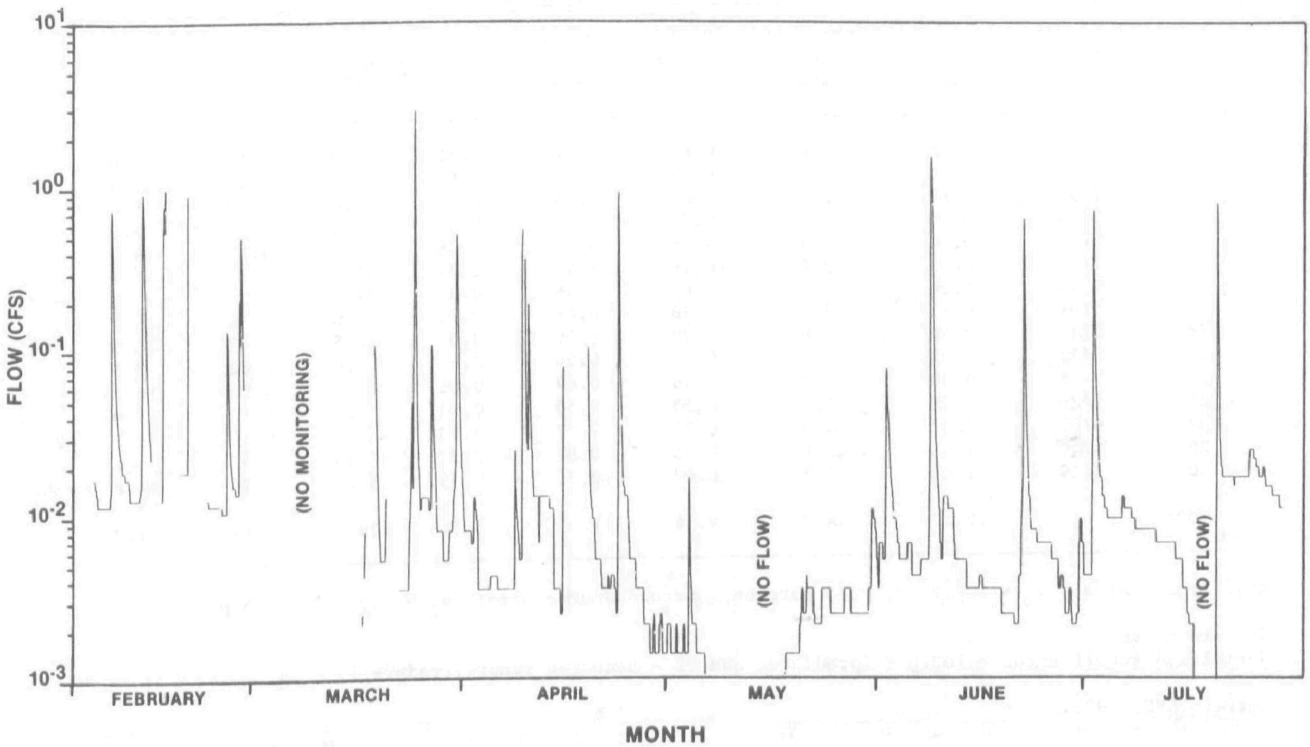

Figure 4. Flow hydrograph at Station H (Harvest Site) — February-July 1983.

Storm*	Beginning date (1983)	Rainfall (in)	Simulation time (days)	Initial storage (in)	Ending storage (in)	Measured runoff		Predicted runoff		Runoff error† (%)
						(in)	(%)	(in)	(%)	
1, 2, 3	2/07	3.71	11.3	1.05	1.28	2.62	70.6	2.61	70.3	0.3
7, 8, 9	3/17	3.95	10.0	1.02	0.98	3.58	90.6	2.77	70.1	-20.5
13,14,15	4/08	4.09	10.0	1.13	1.39	2.57	62.8	2.14	52.3	-10.5
16	4/23	0.90	3.0	1.12	1.27	0.36	40.0	0.388	43.0	3.0
17	5/04	0.50	2.0	0.98	1.03	0.097	19.4	0.132	26.4	7.0
21	5/30	0.50	4.0	0.00	0.00	0.00	0.0	0.010	2.0	2.0
24	6/07	1.70	3.0	0.00	1.06	0.056	3.3	0.235	13.8	10.5
TOTAL		15.35	43.3	5.30	7.01	9.28	60.5	8.30	54.0	-6.4

First Source Area: $\alpha_1 = 37.1$; $P_1 = 30$ percent. Second Source Area: $\alpha_2 = 7.4$; $P_2 = 70$ percent.

*ESE storm number.
†Normalized runoff error = 100.0 * (predicted runoff - measured runoff)/rainfall.

Source: ESE, 1985.

Table 1. Predicted versus measured runoff for Control Site (shrubland).

Storm*	Beginning date (1983)	Rainfall (in)	Simulation time (days)	Initial storage (in)	Ending storage (in)	Measured runoff		Predicted runoff		Runoff error† (%)
						(in)	(%)	(in)	(%)	
1	2/06	1.04	2.0	0.83	1.24	0.50	48.0	0.50	48.0	0.0
2	2/10	0.90	2.5	0.91	1.23	0.65	72.2	0.44	48.9	-23.7
5	2/24	0.65	2.0	0.86	1.15	0.31	47.8	0.23	35.3	-12.5
9	3/24	0.85	2.0	0.99	1.23	0.49	57.6	0.40	47.0	-10.6
10	3/27	0.30	2.5	0.86	0.92	0.08	26.7	0.11	36.7	10.0
11	3/31	1.09	2.0	0.79	1.23	0.34	31.2	0.50	45.9	14.7
16	4/24	1.20	2.5	0.45	0.96	0.35	29.2	0.30	25.0	-4.2
17	5/04	0.51	3.0	0.38	0.46	0.01	2.0	0.08	15.7	13.7
18	5/16	0.40	5.8	0.00	0.00	0.01	2.5	0.05	12.5	10.0
19	5/23	0.25	2.6	0.44	0.38	0.01	4.0	0.02	8.0	4.0
20	5/26	0.30	5.8	0.46	0.07	0.02	6.7	0.05	16.7	10.0
21	5/30	1.18	4.0	0.53	0.58	0.03	2.5	0.14	11.9	9.4
22	6/01	1.10	4.0	0.71	0.77	0.10	9.1	0.40	36.3	27.2
26	6/20	1.35	3.0	0.46	0.88	0.29	21.5	0.33	24.4	2.9
30	7/19	1.05	2.5	0.07	0.35	0.35	33.3	0.26	24.8	-8.5
TOTAL		12.17	46.2	8.74	11.45	3.54	29.0	3.81	31.3	2.2

First Source Area: $\alpha_1 = 2{,}972$; $P_1 = 45$ percent. Second Source Area: $\alpha_2 = 2.97$; $P_2 = 55$ percent.

*ESE storn number.
†Normalized runoff error = 100.0 * (predicted runoff - measured runoff)/rainfall.

Source: ESE, 1985.

Table 2. Predicted versus measured runoff for Harvest Site.

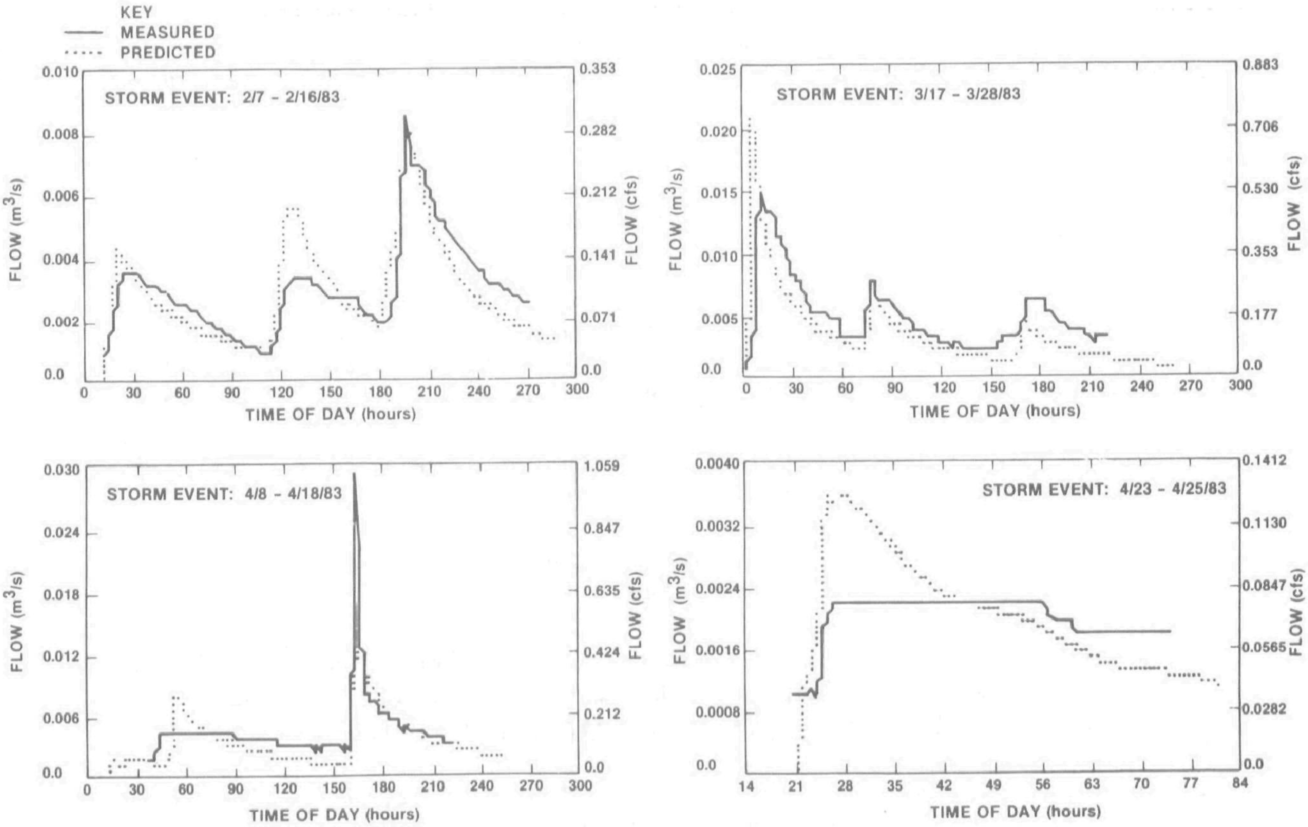

Figure 5. Predicted versus measured runoff for Control Site.

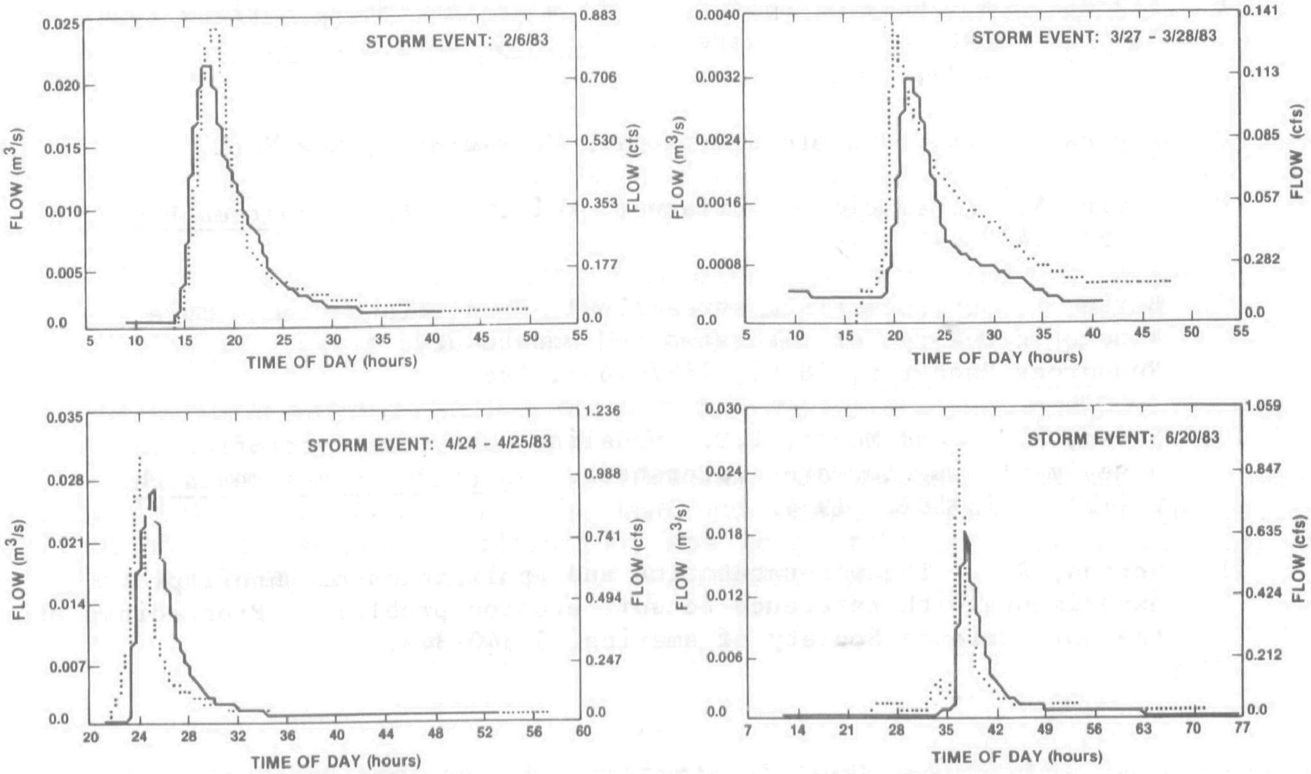

Figure 6. Predicted versus measured runoff for Harvest Site.

Sawchuck, Arch Merritt, and Andrew Middleton of Peat Methanol Associates for their support and assistance. All of the field work was performed by Michael Tomlinson, Kathleen Ingram, Michael Geden, Bill Vogelsong, and others at Environmental Science and Engineering, and we are thankful for their excellent support.

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AREA-WIDE STRATEGIES FOR STORMWATER MANAGEMENT IN SWITZERLAND
CASE STUDY GLATTAL

by: Vladimir[1] Krejci and Willi Gujer
Federal Institute of Water Resources and
Water Pollution Control (EAWAG)
Associated with The Swiss Federal Institutes of Technology
CH-8600 Dubendorf-Zurich, Switzerland

ABSTRACT

An area of 260 km^2 with 230,000 inhabitants was analyzed for the feasibility of storm-water tanks to reduce the pollution during rain events. Due to the number of parameters (sources of pollution, constructed and projected technical measures for water pollution control), the pollution load and the construction and exploitation costs were computed with the help of simulation models.

Simulated results for the "today situation" agreed well with experimental results, thus verifying the simulation models. Then, operating and investment costs as well as residual pollutant load were simulated for different design and operating strategies. Case study results demonstrate the inadequacy of storm-water tanks for cost efficient water pollution control:

- the physical stress on river ecology during rainy weather (flow velocity, bed-load and sediment-transport) can hardly be influenced by the small size tanks utilized in Switzerland.

- the chemical parameters (concentration and load of pollutants in riverwater, DOC, NH_4, PO_4, ...) are only slightly reduced with the exception of substances in street surface runoff (e.g. Pb).

- Storm-water tanks may however help to solve local esthetic problems.

The study demonstrated, that programs which reduce pollutant sources (e.g. lead in gasoline) or slow down and reduce surface runoff (dispersed retention and improved infiltration) are more efficient than a strategy of uniform and area wide application of stormwater tanks.

[1] Visiting Research Engineer at Department of Civil Engineering, University of California in Davis (1984-1985)

INTRODUCTION

Stormwater Management and Control is one of the most complicated aspects of water pollution control. The large number of parameters makes the study of this problem very complex and expensive. Furthermore, conditions within any given watershed vary greatly during wet weather, consequently, local conditions are very important and the results and conclusions are not easily transferred to other areas. This is another reason why research in this area is not very attractive and why most of the studies performed deal with only one part of the much more complex problem.

In Switzerland combined sewer systems (municipal sewage plus stormwater runoff) predominate over separate sewer systems. The hydraulic capacity of wastewater treatment plants is generally twice the maximum dry weather flow. Due to limited hydraulic capacity, excess stormwater is discharged directly to the receiving water. As a result, several times a year untreated, combined sewage is discharged to the receiving water.

In Switzerland two types of stormwater tanks are used as an element of combined sewer systems which have a high frequency of overflow. The first type of tank stores the "first flush" and, after the rain event returns the water to the treatment plant. The second type provides clarification (and some storage) before discharging to the receiving water. The sediments are returned to the treatment plant after the rain ceases.

In Switzerland stormwater tanks are generally concrete; ranging in specific capacity from 15-25 cubic meters per hectare of impermeable surface. They can store approximately 2 mm of rainfall.

The main purpose of this study was to explore the benefits gained from the use of storage tanks in combined sewer systems in relation to the high costs associated with construction of the tanks.

STORMWATER RUNOFF AS WATER POLLUTION PROBLEM FOR WATER COURSES

Studies have shown that during storm events, stormwater runoff results in an increase in the flow and the pollution load in receiving water. The runoff comes from several different sources (Figure 1).

As a consequence of higher flow-velocity, shear stress, transport of sediments, turbidity, change of the concentration of chemical substances and disposal of coarse substances, paper, etc. a stormwater pollution problem arises. The effects of the stormwater runoff can be divided into three areas

- <u>physical (mechanical) effects</u> (flow-velocity, shear stress, transport of sediments, erosion, turbidity) influence the water course as an ecological environment and also affects groundwater infiltration;

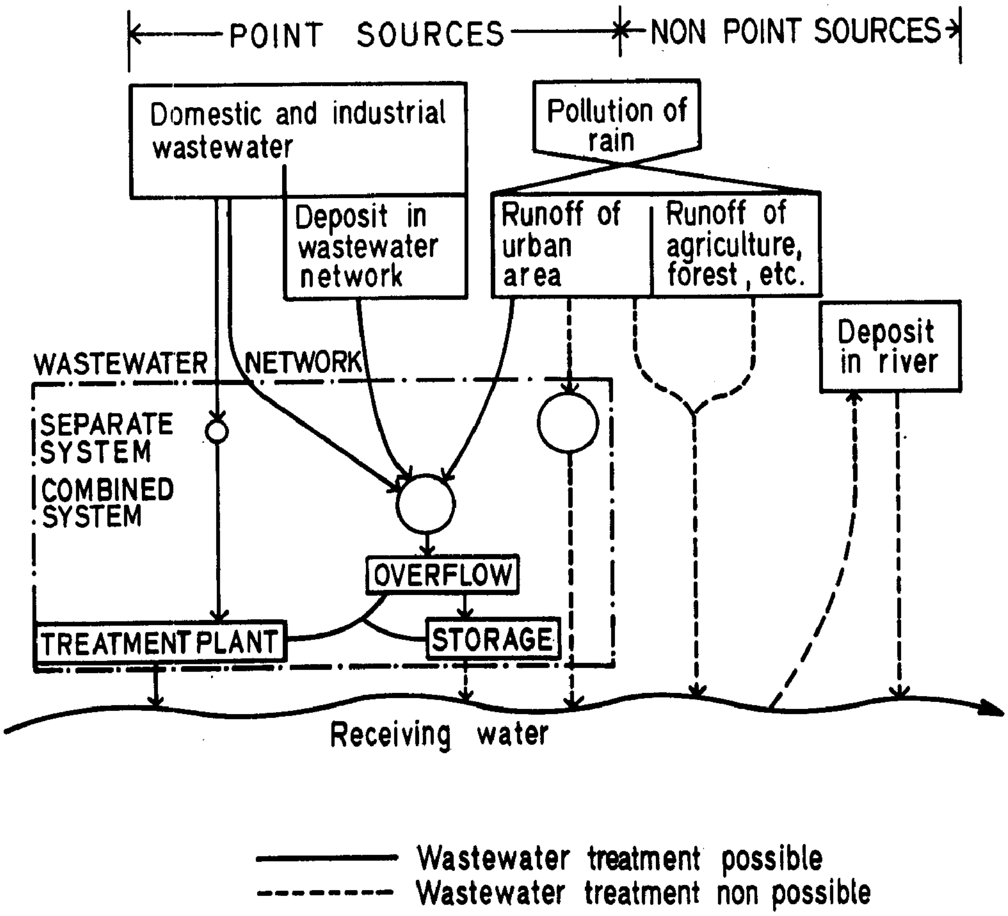

Figure 1. Sources and transport of pollution load during wet weather.

- chemical effects (change of concentration of different chemical substances as DOC, NH_4, Oxygen, Phosphorus, heavy metals, etc.) is significant e.g. for interaction between chemical and biological processes in receiving water, for raw water quality for groundwater-infiltration, drinking water treatment, etc.

- esthetic and health effects (sludge, coarse substances, paper, germs, etc.) can cause local problems (swimming, recreation).

The relative significance of these three aspects of the problem and the results of various methods of stormwater pollution control in Switzerland are demonstrated using the Glatt River Watershed Case Study.

GLATTAL-AREA

The Glatt is a small river in northeastern Switzerland. The river has an average discharge ranging from 3.3 m^3/s at the point of outflow from Lake Greifensee to 8.7 m^3/s at the point of confluence with the Rhine. The watershed is approximately 260 km^2, and is heavily urbanized (about 900 persons/km^2). About 40 km^2 (16% of total area) are impervious surfaces. There are 13 wastewater treatment plants in the area and nearly 100% of the wastewater receives secondary treatment (Figure 2).

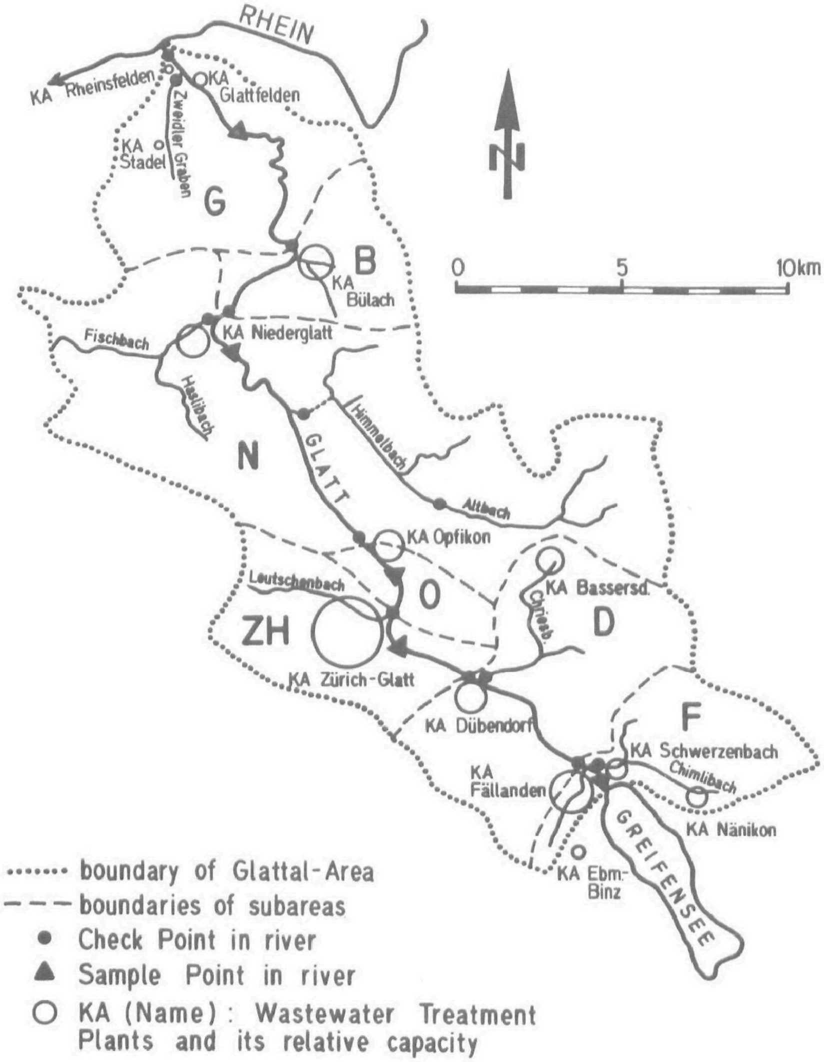

Figure 2. Glatt River Watershed (Switzerland). Survey of catchment of River Glatt with the important tributaries and survey of the existing wastewater treatment plants in the areas.

In spite of nearly complete wastewater treatment the quality of the Glatt River is poor. In 1976, the emphasis of Swiss regulation changed from one of setting effluent standards to one of regulating receiving water quality. With regard to esthetic, biological and chemical criteria, the Glatt River does not satisfy the standards set by Swiss Law.

In response to this situation, a two phase study was initiated to determine possible pollution control options. The first phase of the study centered on upgrading dry weather treatment to the tertiary level (nitrification, chemical precipitation and coagulation, partly also filtration). The second phase centered on possible stormwater runoff pollution control measures. During the second phase, four major questions were:

1. What is the volume and duration of stormwater runoff in the Glatt River and in its important tributaries?

2. How large are the pollution loads from the point and non-point sources during stormwater runoff?

3. What are the effects and the costs of stormwater pollution control measures, especially those of overflow storage tanks in a combined wastewater network?

4. What is the priority of different measures of water pollution control (dry weather and stormweather) in this area?

THE EFFECT OF WATER RUNOFF ON RECEIVING WATER IN THE GLATT RIVER

The effect of stormwater runoff on the Glatt River are based on detailed analyses of:

- basic hydrologic conditions;

- wastewater collection networks;

- wastewater treatment plants in the area; and

- results of chemical and biological analysis in sewers, treatment plants, and receiving waters.

The costs of management alternatives and the resulting pollution loads were calculated with an EAWAG simulation model and the results verified against existing data. The verification and the sensitivity analysis proved the feasibility of the simulation model to answer the questions posed above.

PHYSICAL EFFECTS

As a consequence of agricultural drainage, channelization of the Glatt and its tributaries, and development of urban and traffic areas, the discharge and the flow-velocity in the Glatt is significantly increased during wet weather periods.

Figure 3 shows the distribution of discharge and flow-velocity in the river Glatt over one year. Several times a year the river bed is loosened, resulting in drastic changes in the environment for the river biota as well as infiltration into the groundwater, etc.

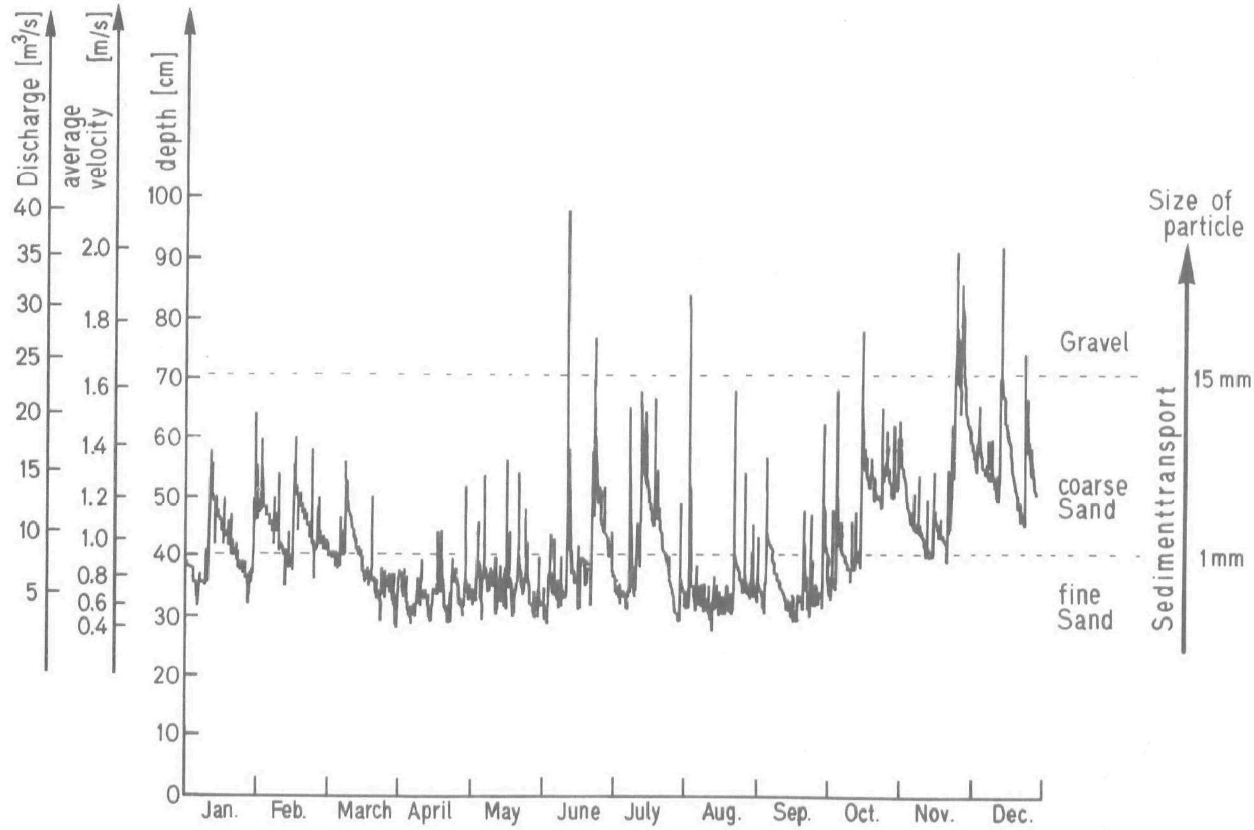

Figure 3. Depth, flow velocity, discharge and the approximate sediment transport in the river Glatt (1974).

The relative significance of the different sources of the flow and turbidity in the river Glatt during wet weather is shown in Table 1. As can be seen, the greater part is derived from non-point sources.

TABLE 1. SIGNIFICANCE OF THE DIFFERENT SOURCES OF THE FLOW AND TSS LOAD (TURBIDITY) IN THE RIVER GLATT DURING WET WEATHER

Source of Load	average load during wet weather in % Flow	TSS
Wastewater treatment plants	22	<10
Stormwater overflows (combined syst.)	10	15-25
Stormwater runoff (separative syst.)	6	<10
Non point sources	62	>50

"The results of this study showed that during stormwater runoff the "ecological systems" in the river suffer from considerable physical stress. The high velocity of the flow causes an interruption of natural development and an unnatural selection of river-biota.

The high erosion of agricultural surfaces and the runoff from the impermeable urban and traffic areas causes a high turbidity in the receiving water and influences sunlight penetration in the river (Figure 4).

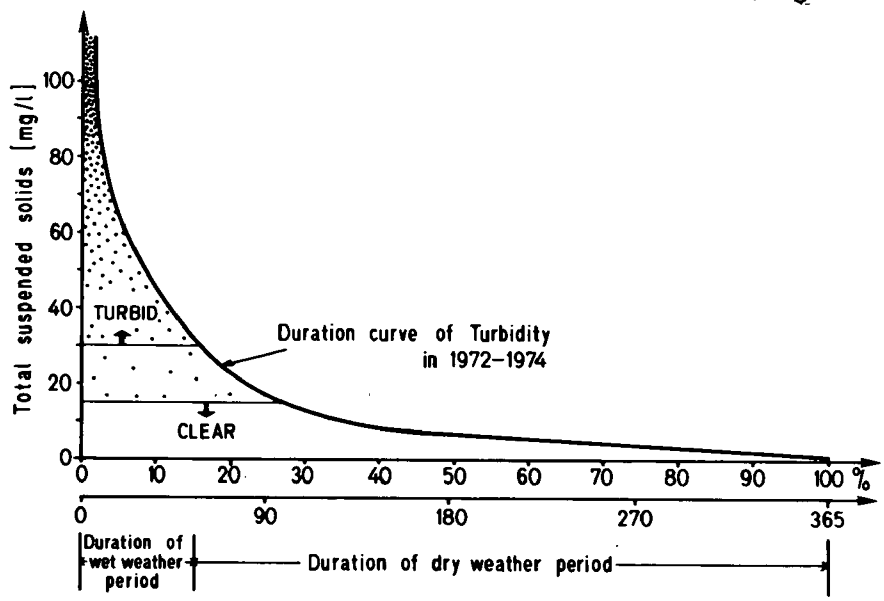

Figure 4. Duration curve of concentration of suspended solids (TSS) in Chriesbach (tributary of Glatt River).

CHEMICAL EFFECTS

The following chemical parameters were used to represent groups of chemical substances:

Dissolved Organic Carbon (DOC) - represents dissolved chemical substances with their origins from domestic and industrial wastewater, runoff from impermeable urban areas, and non-point sources;

Ammonium - represents dissolved substances with their origins mainly from domestic wastewater;

Total-Phosphorus - represents partly dissolved substances from domestic wastewater;

Lead - represents small particulate substances from the runoff from impermeable urban and traffic areas.

Figure 5 shows the relative load of different sources during 13 hypothetical rains which statistically represent the various storm events experienced in one year. The high relativer load from point sources is striking.

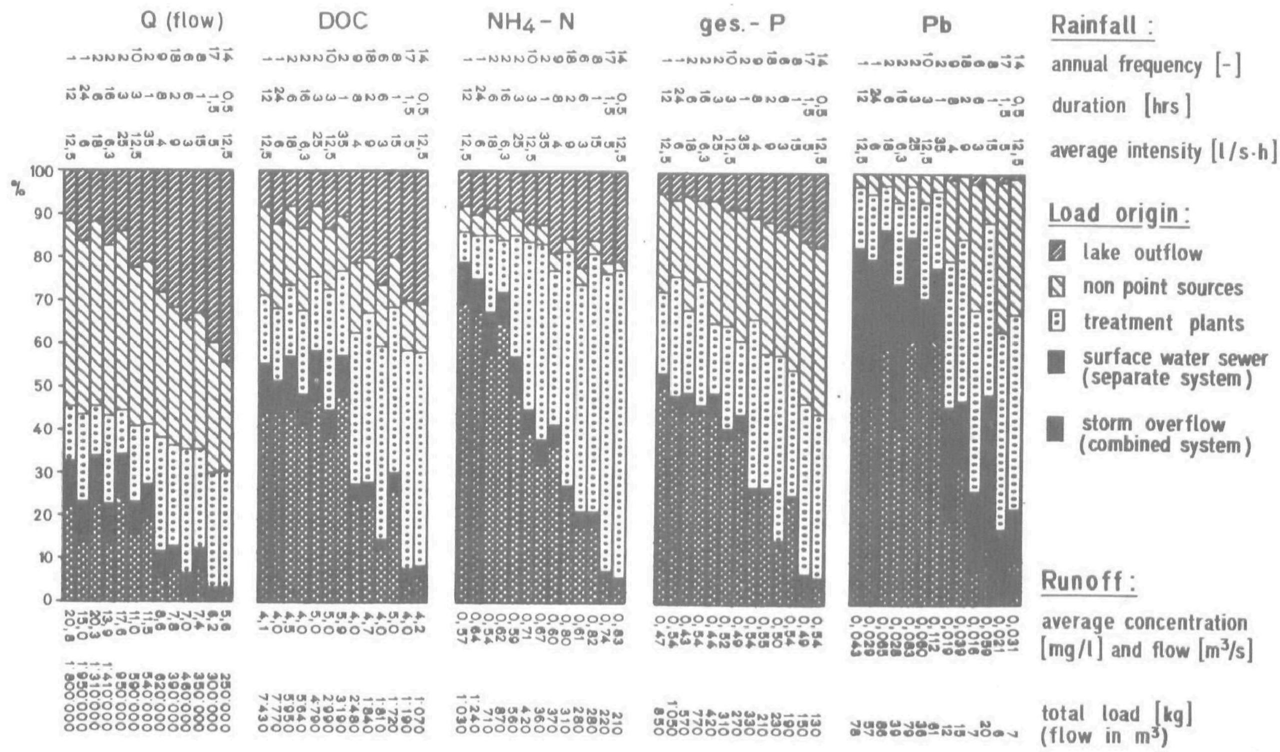

Figure 5. Calculated wet weather pollutant load and concentration in the river Glatt during 13 different rainfalls (percentage from important sources).

Figure 6 shows the annual pollution load in the Glatt River versus the Chriesbach River. The Chriesbach River was chosen for comparison due to its small watershed area (approximately 16 km^2) as compared to the Glatt River. The Chriesbach area is also heavily urbanized. In both Rivers, dry weather sources represent the major sources of the selected parameters, except lead.

Figure 7 shows typical flow concentration changes with time during a summer storm event in the river Glatt. The concentration of particulate substances (represented by TSS and heavy metals tied to TSS) increase greatly. The concentration of substances from domestic and industrial wastewater (represented by DOC, NH_4-N, and P) stayed nearly constant, some other substances (Cl^-, Ca^{2+}, Mg^{2+}) were diluted.

We can assume that these relatively small, short-term changes in concentration of DOC, ammonium, nitrate and phosphorus have negligible effects on water quality. However, the concentrations of heavy metals reach significantly higher values than during the dry weather period. Although the greater part of the heavy metals load is in particulate form there may still be toxic effects.

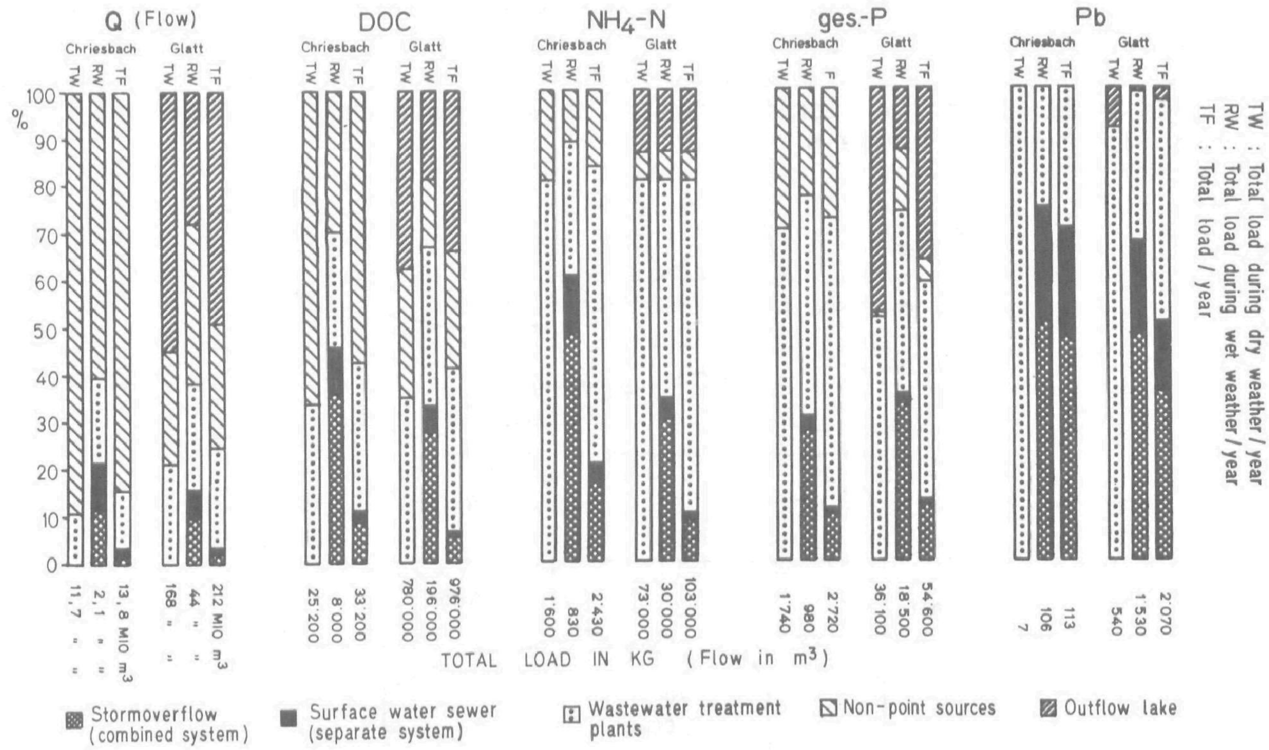

Figure 6. Calculated annual load in the river Glatt and in Chriesbach (tributary of the Glatt), percentage from important sources.

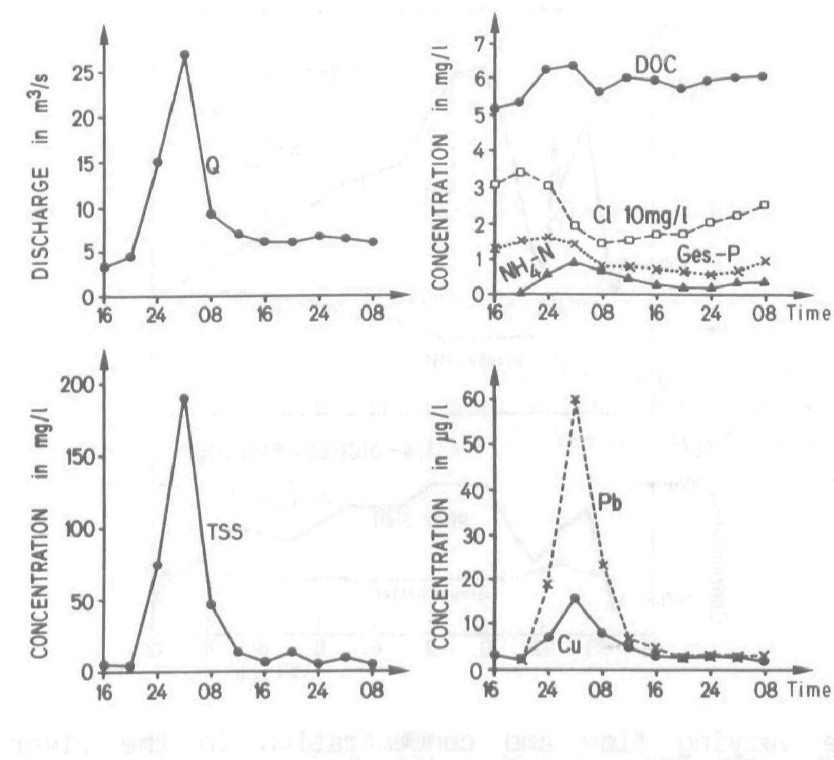

Figure 7. Typical flow and concentration charges with time during a summer storm event in the river Glatt (July, 19-20, 1976).

During wet weather periods other substances also get into receiving water (e.g. many different hydrocarbons) from different sources. We could not estimate the amount of these substances with the simulation model. However, we have some hints about the significance of these substances in groundwater, for example, as shown in Figure 8.

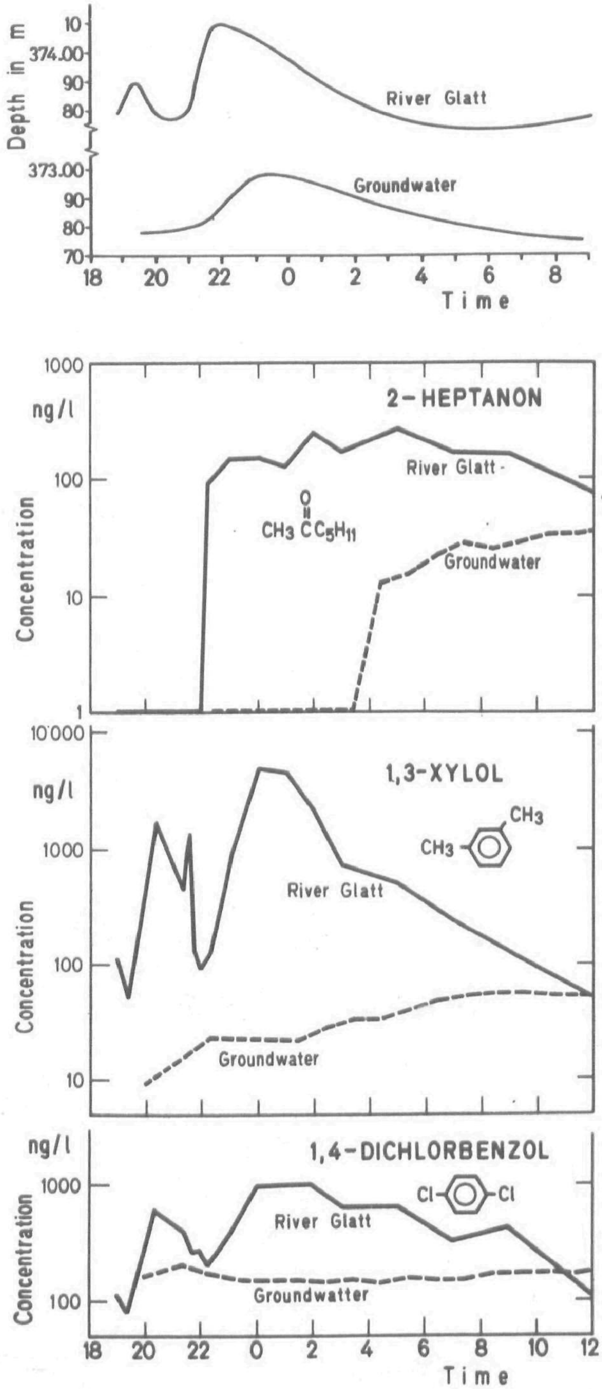

Figure 8. Time varying flow and concentration in the river Glatt and in groundwater (distance of groundwater sampling to river bank ~2.5m) during stormwater runoff.

Another shortcoming of the model is it's inability to accurately model suspended solids (TSS) and particulate organic carbon (POC), due to the lack of necessary basic information (the load from non point sources versus the load from deposits in sewer lines).

The short-term change in dissolved oxygen concentration in the river Glatt from 9-10 mg O_2/l to 4-6 mg O_2/l several times a year during a stormwater event may result from the DOC and POC load during wet weather. However, the evidence is not conclusive. On one hand, many easily decomposable organic substances reach the receiving water from the sewage overflows; on the other hand the anaerobic deposits in the river are stirred up. It was not possible to simulate this problem.

ESTHETIC EFFECTS

The physical and chemical loads during stormwater runoff influence large areas of the water course. Esthetic problems occur only in the local area (in the vicinity of outfalls). In the Glatt-Area we are aware of only a few local problems.

POSSIBLE MEASURES FOR STORMWATER POLLUTION CONTROL

The effects of four different stormwater pollution control strategies and their costs were investigated:

- reduction of pollution load from individual sources;
- changes in the character of runoff;
- treatment of storm overflow water (in the combined wastewater network); and
- enlargement of the hydraulic capacity of wastewater treatment plants to accommodate wet weather flows.

REDUCTION OF POLLUTION LOAD FROM INDIVIDUAL SOURCES

Table 2 demonstrates, that the substitution of phosphorus in detergents and the removal of lead from gasoline have about the same effect on water quality in the River Glatt as a very large increase in storage of stormwater overflow.

TABLE 2. POLLUTANT LOAD DURING WET WEATHER: OVERFLOW STORAGE VS. REDUCTION OF POLLUTION FROM INDIVIDUAL SOURCES

RAINFALL: average intensity: 12.5 ℓ/s·ha duration: 3 hrs	Phosphorus and Lead Load in River Glatt in kg			
PHOSPHORUS				
Phosphorus in detergents	yes		no	
volume of overflow-storage m^3/ha^*	0	35	0	35
Total Load	306	281	205	191
Load of overflow	111	66	59	32
Outflow of treatment plants	79	100	31	43
LEAD				
Lead in Gasoline	yes		no	
volume of overflow-storage m^3/ha^*	0	35	0	35
Total Load	46	34	5.2	4.3
Load of overflow	34	18	1.8	0.7
Load of separate sewers	6.5	6.5	0.0	0.0

* of impermeable surface

CHANGES IN THE CHARACTER OF RUNOFF

There are several possibilities for changing runoff:

1. The impermeable surface can be reduced (for example if stormwater from roofs and parking areas is infiltrated). Table 3 shows the effect of a 20% reduction in the runoff-coefficient. Despite the reduction in pollution load, a relatively larger part of the residual stormwater will be treated in the wastewater treatment plants.

2. A periodical flooding of flat roofs, not intensively used traffic surfaces, and parking areas, and the use of storage capacity in the sewage system could retard the runoff-discharge. The effect of these changes can not be calculated yet, but it is certain that all retardation measures have a positive influence on the receiving water.

TABLE 3. POLLUTANT LOAD DURING WET WEATHER: OVERFLOW STORAGE VS. REDUCTION IN THE RUNOFF-COEFFICIENT

RAINFALL: average intensity: 12.5 ℓ/s·ha duration: 3 hrs					
Runoff-Coefficients		Existing runoff		Diminuation of impermeable surface of 20%	
Volume of overflow-storage m^3/ha*		0	35	0	35
Total Load					
Q	m^3	594 000	594 000	560 000	560 000
DOC	kg	3 168	2 760	2 820	2 420
NH_4-N	kg	395	365	363	332
Tot.P	kg	306	281	285	260
Lead	kg	46	34	36	25
Load of overflow					
Q	m^3	92 500	59 400	70 700	37 000
DOC	kg	1 420	802	1 125	513
NH_4-N	kg	160	102	133	73
Tot.P	kg	111	66	93	46
Lead	kg	34	18	26	11

* of impermeable surface

3. The drainage of urban areas through separate systems instead of combined systems would significantly reduce pollution in receiving waters (Table 4).

No one of these measures could be realized in a short time. However, long-term, systematical inclusion of these measures during the process of renewing old systems will result in a significant contribution to the protection of receiving water.

TREATMENT OF OVERFLOWS

There are also several possibilities for the treatment of stormwater overflows. All these alternatives tend to eliminate coarse particulate substances. Dissolved substances (DOC, NH_4-N and a large portion of the phosphorus) can be reduced only through the storage and subsequent treatment.

For the treatment of stormwater overflow in Switzerland different kinds of storage tanks are used. These tanks can significantly affect the pollution load from stormwater overflows, (depending on rainfall, storage tank volume, etc.). However, the effect on the total pollution load in the receiving water is minor (Figure 9 and 10).

TABLE 4. POLLUTANT LOAD DURING WET WEATHER: OVERFLOW STORAGE VS. SEPARATE SEWAGE SYSTEM

RAINFALL: Average intensity: 12.5 ℓ/s·ha duration: 3 hrs								
separate system in the area	existing situation (~ 25%)				100%			
volume of overflow-storage m^3/ha*	35				None			
Parameter	DOC	NH_4-N	Tot.P	Pb	DOC	NH_4-N	Tot.P	Pb
Total Load kg	2764	365	281	36	2061	226	193	26
Load of overflows	802	102	66	19	--	--	--	--
of separate system	193	23	6	6	693	82	23	23
outflow of treatment plants	949	172	100	6	549	75	61	1
of non-point sources (agriculture, etc)	819	69	109	2	819	69	109	2

* of impermeable surface

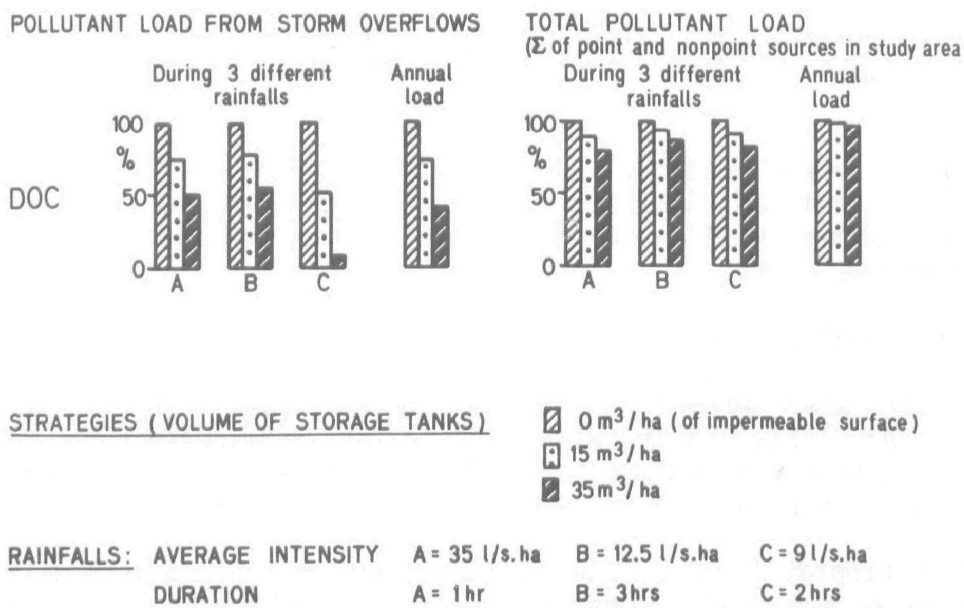

Figure 9. Effect of stormwater overflow storage tanks reduction of DOC-Load in the river Glatt (similar effects for NH_4-N, Tot. P and Lead).

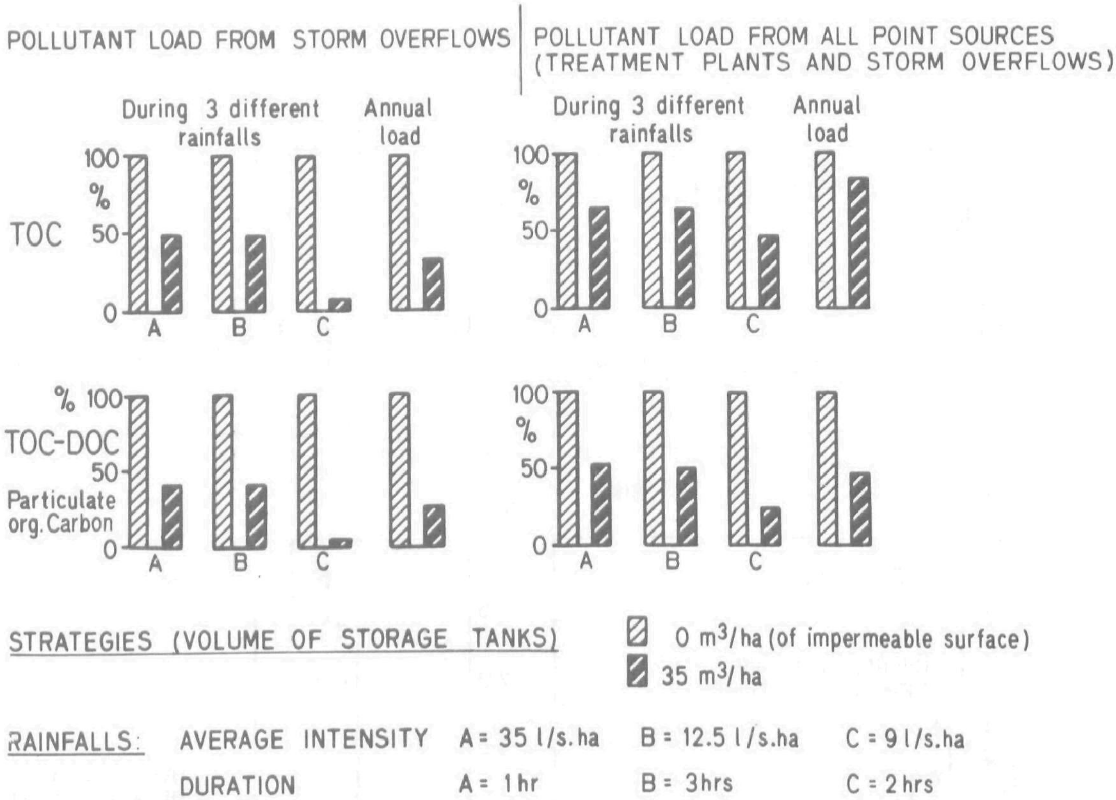

Figure 10. Effect of stormwater overflow storage tanks on reduction of TOC and POC load in point sources (wastewater network and plants) in the Glatt River Watershed (particular org. carbon (POC) = TOC-DOC).

ENLARGEMENT OF THE HYDRAULIC CAPACITY OF WASTEWATER TREATMENT PLANTS

The hydraulic capacity of wastewater treatment plants in Switzerland is generally twice the maximum dry weather flow. An enlargement of the hydraulic capacity of treatment plants requires an enlargement of their primary and secondary settling tanks.

Calculation of reduction in pollution load versus the costs of enlargement shows, that in watersheds with a small specific dry wastewater flow per unit of impermeable surface (for example in rural areas), storage tanks are more cost-effective than enlargement of treatment plants. For the large city areas the opposite is true.

In treatment plants with a capacity in the primary settling tanks of 3-5 times the dry weather peak, and with overflow after mechanical treatment, a good removal of particulate substances can be achieved. However, at the same time a higher load of dissolved substances will be displaced from the primary settling tanks to the receiving water (Figure 11).

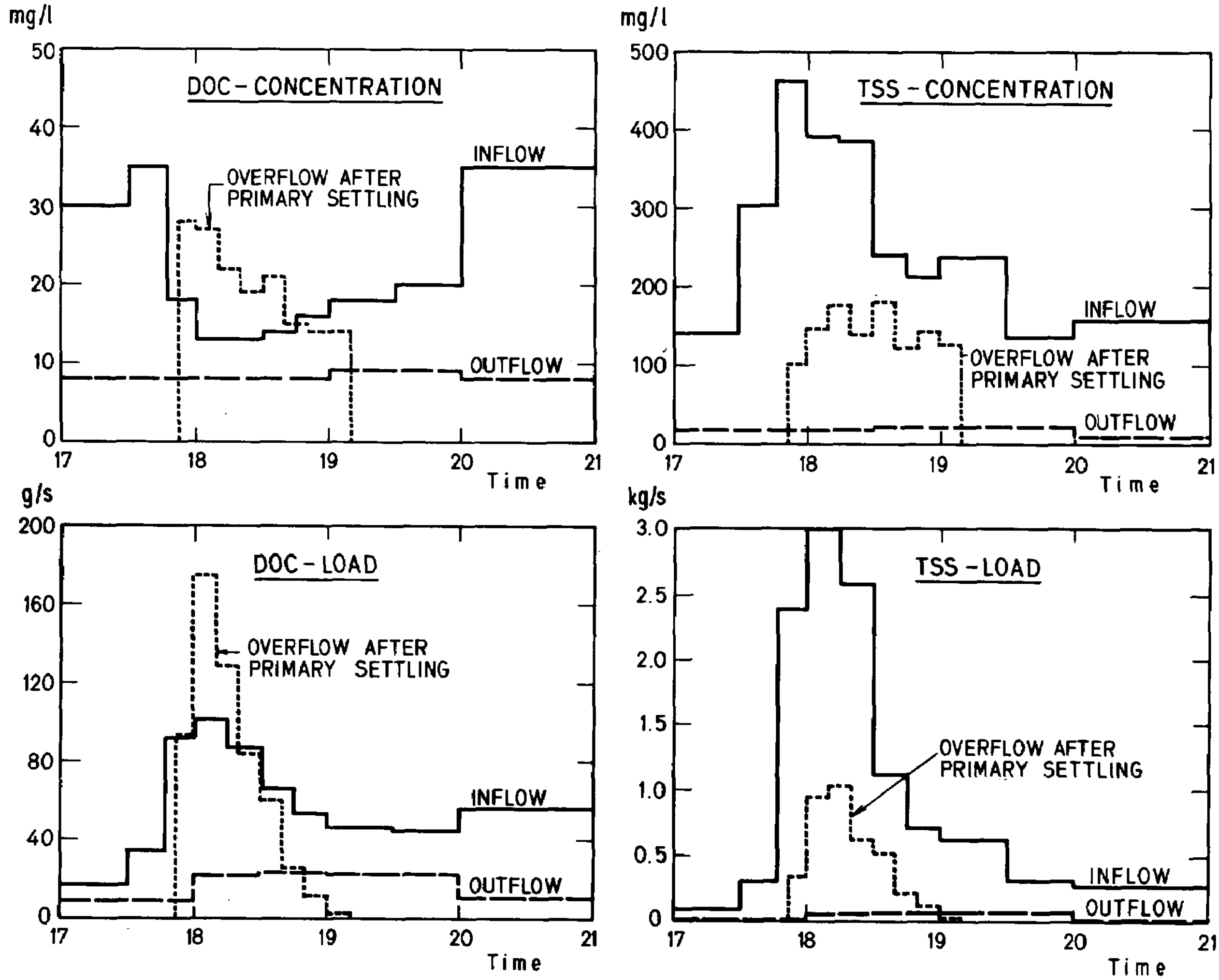

Figure 11. In treatment plants with a primary settling tank capacity 3-5 times dry weather peak and with overflow after mechanical treatment a good removal of particulate substances can be achieved. At the same time a higher load of dissolved substances will be from the primary settling tanks.

COST EFFECTIVENESS OF DRY WEATHER VS WET WEATHER POLLUTION CONTROL

Table 5 shows the costs and the resulting load reduction of wastewater treatment plants versus storage tanks in the Glatt River area.

The existing treatment plants are designed for removal of the suspended solids and organic substances (secondary treatment). The proposal for advanced wastewater treatment in the area will bring a relatively efficient and favorable removal of ammonium and phosphorus.

TABLE 5. THE COSTS AND EFFECTS OF WWTP VS. STORMWATER OVERFLOW STORAGE

MEASURES	Cost of investment	Cost annual	Removal in t/a				Specific costs* Fr/kg			
	Mio Fr.	Mio Fr.	TSS	DOC	NH_4-N	Tot.P	TSS	DOC	NH_4-N	Tot.P
Existing treatment	122	9.9	9000	1570	180	110	1.1	6	55	90
Extended	47	3.6	350	150	365	240	10	24	11	15
TOTAL	169	13.5	9350	1720	545	350	1.4	7.8	25	39
Stormwater storage tanks										
15 m^3/ha**	12	0.6	220	10	2.0	1.4	2.7	60	300	400
35 m^3/ha**	26	1.3	500	22	4.4	3.7	2.6	60	300	400

* specific costs: 1 Fr = 0.5 U.S. $
** specific volume of an impermeable area

SUMMARY AND CONCLUSIONS

Table 6 shows the different measures and their effects on stormwater pollution control.

The table shows, that we can not decrease the physical/mechanical effects with conventional measures. However, we have the possibility of reducing both the chemical as well as the esthetic effects on receiving waters.

In many water courses the physical effects during stormwater runoff cause the most important stress on biota in the receiving water. Conventional measures in the sewage network and in treatment plants are not suitable to reduce this stress.

In practical terms, control of pollution from stormwater runoff has two aspects:

1) chemical loads and resulting effects which impact the entire reach of the river; and

2) local problems (well defined esthetic and biological effects of sludge, sediments and coarse substances).

The chemical load during stormwater runoff in Switzerland could be significantly reduced with long-term measures (elimination of causes, retention, infiltration, etc). All these long-term measures also have a significant impact on the esthetic effects and partly reduce the physical stress.

TABLE 6. EFFECT OF DIFFERENT MEASURES ON THE REDUCTION OF POLLUTANT LOAD IN WATER COURSES DURING STORMWATER RUNOFF

Measures	Effect of Pollution Physical	Chemical	Esthetic
Reduction of Pollution Load from individual sources (P,Pb)	0	+++	0
Changes in the character of Runoff			
- Retardation (flooding)	+	+	+
- Storage tanks	0	+	++
- Reduction of impermeable surfaces	+	+	+
- Installing separate sewer system	0	+(-)	++
Treatment of stormwater runoff			
- Enlargement of treatment plants	0	+	+
- Treatment of overflow	0	0	++

\+ positive effect
0 no effect
\- negative effect

The local problems will have to be identified and defined, then solved with specific and suitable measures.

ACKNOWLEDGMENTS

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THE IMPACT OF "SNOW" ADDITION ON WATERSHED ANALYSIS USING HSPF

by: Stan Udhiri[1]
Mow-Soung Cheng[1]
Richard L. Powell[1]

ABSTRACT

The hydrological Simulation Program - FORTRAN (HSPF) is a powerful tool for in-depth analyses of the hydrologic and associated water quality processes in a river basin. The model use requires a thorough understanding of the various algorithms that make-up the model. One of the most complicated algorithms is the section referred to as "SNOW"; not only because of the massive amount of calculations involved, but also due to the several additional meteorologic time series which are usually not easy to obtain. The "SNOW" section in the HSPF model simulates the hydrologic processes involved in the accumulation and melting of snow and ice on a land segment. It is important, as a significant amount of runoff is derived from snow, especially in the northern part of the United States.

The model was used to assess the hydrologic and water quality impact of planned development within Piscataway Creek in southern Prince George's County, Maryland. Prince George's County is located in the Washington Metropolitan area which has a humid, continental type of climate with the temperatures falling below the freezing point, approximately 90 days each year. During the study, one of the questions dealt with was what effect the inclusion of the "SNOW" option would have had on the study result. To address this question, the following four cases were studied: (1) Develop and calibrate the model without SNOW; (2) Add SNOW to case (1) and calibrate SNOW parameters only; (3) Develop and calibrate model including SNOW option; and (4) After the model in case (3) is well calibrated, remove SNOW calculation.

[1] Water Resources Engineers, Maryland-National Capital Park and Planning Commission, 14741 Governor Oden Bowie Drive, County Administration Building, Upper Marlboro, MD 20772.

The results of the analyses, including both aspects of hydrology and water quality, for all the above four cases are presented here.

INTRODUCTION

This study is intended to evaluate the impact of snow/ice accumulation and melting processes on Piscataway Creek in the Washington Metropolitan Area using the HSPF model. Piscataway Creek is located in Prince George's County, Maryland. The Piscataway Creek Study was originally undertaken by the Metropolitan Washington Council of Governments (COG) in 1983 (Reference 1) to investigate the stream's water quality and to identify stormwater and nonpoint source management needs within the watershed. The COG watershed model was developed as a water resource planning tool for the evaluation of land use planning alternatives and applied management practices, and for the assessment of their impacts on stormwater runoff and water quality. However, the effects of accumulation and melting of snow/ice were ignored in the COG study.

From long-term statistics, the average number of days that the temperature is 32°F or less in the watershed is approximately 90 days every year (Reference 2). For an area where 25 percent of the year the temperature is 32° F or below, the snow/ice accumulation and melting might be of some significance. It is known that from a hydrologic point of view, the major difference between snow and rain is that snow is not transformed to surface runoff immediately resulting in (1) a less peak flow rate and a later peak time; (2) a higher surface detention capacity; and (3) a higher soil moisture content when snow melts. Furthermore, the phenomena of surface pollutant accumulation and removal/washoff are vastly different between snow and rain. For example, most of the nonpoint pollutants are transported from the land surface in the early portion of rainfall events; while most of the pollutants remains on the land surface during a snow storm (covered by snow) until the accumulated snow starts to melt. Also, a snow cover reduces soil detachment and washoff and therefore reduces soil loss.

On the basis of a possible significant effect on "SNOW" addition, it was decided that a sensitivity analysis on the effects of snow/ice acumulation and melting is important. Four cases were involved in this study. The first case is to recreate COG's Piscataway Creek Watershed model using the same parameters, meteorologic time series, and input sequence as those of COG. In the second case, "SNOW" related parameters and time series were added to the input sequence of case 1 and Time Series Store (TSS) files. At this point in time, only "SNOW" related parameters were adjusted for calibration purposes. Other parameters were kept the same. The next case (case 3) was the adjustment of other parameters as well. For example, monthly upper zone storages were modified to a more realistic profile; INFIL was reduced; Manning coefficients were changed to reflect the seasonal effects on vegetation, etc. The final case (case 4) was the removal of "SNOW" calculation from case 3. The primary difference between case 1 and case 4 is that the "monthly upper zone storages" for the winter period were manually increased in case 1 to reflect the effect of snow accumulation on the land segment, although the snow accumulation might not always be on the ground for the entire winter period. As a result, the simulated storm runoff for case 1 would be smaller than that for case 4 in the winter. The results from the four cases were then analyzed.

The Piscataway Creek watershed encompasses a complex mixture of land uses from heavily urbanized areas at the upstream portion of the watershed to rural, forested and agricultural areas downstream. Current land uses in the watershed include: agricultural, which occupies 15 percent, residential and commercial developments at 15 percent, with the rest of the watershed primarily forest and open unforested land.

Piscataway Creek has a relatively well-defined valley, with channel elevations ranging from sea level to 250 feet above mean sea level. The watershed lies within the Atlantic Coastal Plain Province. Soils are principally sandy loams and silt loams derived from the underlying sedimentary materials (Reference 3). The watershed has a humid, continental type of climate by reason of over 40 inches of average annual precipitation and its location in the middle latitudes where the general atmospheric flow is from west to east across the North American continent. The long-term mean annual temperature in the watershed is approximately 54° F (Reference 4).

GENERAL MODELING PROCEDURES

The Hydrological Simulation Program views the processes within the hydrologic cycle as a series of mathematical representations with the physical characteristics of the watershed entered as parameters. The programming is divided into four load modules: TSSMGR, PERLND, IMPLND, and RCHRES. TSSMGR performs data management for any time series stored in the system. PERLND and IMPLND simulate snowpack and soil profile processes and calculate continuous soil moisture, evapotranspiration, groundwater accretion, sediment detachment and washoff, soil and water temperatures, and water quality constituents generation. RCHRES assembles and routes the information (inflow) determined by PERLND and IMPLND, as well as certain lateral inflows and outflows, through the drainage network, including reservoirs.

Values of several required parameters for PERLND, IMPLND, and RCHRES can be determined directly from physical characteristics of the watershed. The parameter values for land slope and land cover, for example, can be determined directly from topographic and land use maps. FTABLES for channel or reservoir routines can be calculated though hydraulic analyses from channel geometry. However, values for some parameters, such as nominal soil moisture storage and infiltration rates are usually obtained through the calibration process by first assuming an initial value for each of these parameters. The simulated streamflows and water quality constituents are compared with observed values at sampling sites. If there is a substantial difference between the simulated and recorded data, the parameters have to be changed and the procedure repeated.

The explicit purpose of model calibration is to adapt the HSPF model to the hydrologic, meteorologic and physiographic characteristics of the watershed. The process essentially entails the adjustment and fitting of varius parameters and constants used in the mathematical expressions. This fitting procedure begins with the "LAND" phase (PERLND and IMPLND) of the hydrologic cycle and then proceeds through the channel/reservoir routines.

MODEL INPUT DATA

The application of HSPF watershed model requires a great deal of data. This data, which will be discussed in detail in subsequent sections, were scrutinized before input to the model and then re-evaluated as unusual results developed during the calibration process. This data includes meteorologic time series and steady state data.

Meteorological time series data are critical inputs for both hydrologic and water quality simulation. All hydrologic simulations of runoff require precipitation and potential evapotranspiration data. Hydrologic studies which simulate snowmelt and water quality studies require additional time series data for air temperature, wind speed, solar radiation, cloud cover, and dewpoint temperature. Plankton simulation requires solar radiation data. Wind speed may be required for simulation of dissolved oxygen (Reference 5). The detailed description of the meteorological time series is provided in Reference 6.

The steady state data include land usage (land cover), soil type, surface slope, channel slope, and channel geometry. These data are used to establish the values of the model parameters. If the physical characteristics of the watershed remain the same, the parameters are constant over the simulation period. However, if substantial changes in physical characteristics, such as land uses, occurred during the simulation period the corresponding changes would be made in the model to reflect the changes. The detailed information of the steady state data is also described in Reference 6.

The entire 39.5 square miles of drainage above the USGS stream gauge at Piscataway was divided into 7 channel reaches. Each reach represented an idealized drainage path through a tributary area and was assumed to have relatively constant hydraulic properties through its length. Each reach could accept: runoff from its tributary area;

inflow from upstream reaches; and municipal and/or industrial discharges, if any. At the same time, both consumptive and nonconsumptive withdrawals might occur from each reach to simulate irrigation or water supply. The resulting net runoff or water pollutants from each reach was then discharged to the next downstream reach. A function table (FTABLE) was created using the geometric data of each channel reach with the Manning Equation. The FTABLE is a HSPF input used to represent the hydraulic and geometric characteristics of each reach.

CALIBRATION RESULTS

The general calibration procedures of the HSPF model are relatively standardized. The detailed procedures and guidelines are presented in Reference 5 and are not repeated here. The following sections present the calibration results from each of the four cases.

SOURCES OF ERROR

The ability of the hydrologic model to produce results that correlate well with the recorded data involves three considerations: model error, data error, and calibration error. Model error results when one or more of the mathematical relationships do not adequately describe the prototype process. The results of numerous applications and tests indicate that this error for the HSPF model is insignificant.

Data errors are of two types: measurement and random. Measurement error occurs when the recording instrument malfunctions or the observer misreads the instrument and no value (or an incorrect value) is recorded. Every attempt has been made to correct these errors but some residual errors always remain. Random errors occur in may ways. The most common random errors occur in the measurement of climate conditions and particularly in the measurement of precipitation. While these discrepancies are termed "errors", they are actually variations which do in fact occur in the real world. A single rain gauge rarely represents the true precipitation over a watershed. Even the use of multiple rain gauges does not insure precise representation of the spatial and temporal variations of rainfall and snow. The hourly precipitation time series used in this study was developed through the application of the National Weather Service Mean Areal Precipitation program (Reference 7) using the recorded data at thirteen nearby gauging stations. This composited hourly precipitation time series might not truly represent the actual precipitation for the entire watershed. Also, the stage-discharge relationship for the stream flow gauging station might not accurate, especially during high flood stages. For example, the maximum discharge of 5,000 cfs recorded at the Piscataway Creek gauge on September 26, 1975 was estimated from rating curve extended above 1700 cfs. Furthermore, required solar radiation information is not available for all areas of the State of Maryland. Several assumptions were therefore necessary to generate this information from "cloud cover" data recorded at Baltimore-Washington International Airport. Some errors might have been introduced into the model, from this data.

Calibration error occurs when incorrect parameter values are chosen. Such an error can lead to persistent bias where simulated flows are too high or too low, or it can lead to sporadic bias when the conditions of an infrequent phenomenon are misrepresented. Calibration error can be minimized by comparing simulated and observed information which has short time intervals but long-term simulation periods.

Very often in river basin simulation, a great deal of time is expended in an effort to match recorded data which are in themselves questionable. Therefore, at the onset of this calibration process, the integrity of all the data was reviewed. The two data series of prime importance in any hydrologic model are obviously precipitation and stream flow. Just as obvious is the need to scrutinize these records thoroughly so that weak or misleading data can be identified.

Figure 1 depicts the spatial variation in the monthly precipitation for September 1975, over Prince George's County, Maryland. The isohyetal lines in this figure were drawn from the information recorded by more than 50 voluntary observers. If fewer or more gauges had been used, the isohyetal would have been presented differently. This is a result of the variation in precipitation which exists over the watershed. Thus, it can be seen that an infinite number of rain gauges would be required for the definition of the spatial and temporal variation. This, of course, is practically infeasible.

The accuracy of streamflow data depends primarily on (1) the stability and accuracy of the stage-discharge relationship, and (2) the accuracy of observations of stage and the interpretation of records. The USGS rates their streamflow data according to the following criteria:

Excellent = about 95 percent of the daily discharges are within 5 percent of the true values

Good = within 10 percent

Fair = within 15 percent

Poor = less than "Fair" accuracy

The accuracy ratings of the Piscataway Creek USGS gauge used for the calibration-verification period are reported as "Fair", in general. With this stream flow rating and the spatial variation in precipita-

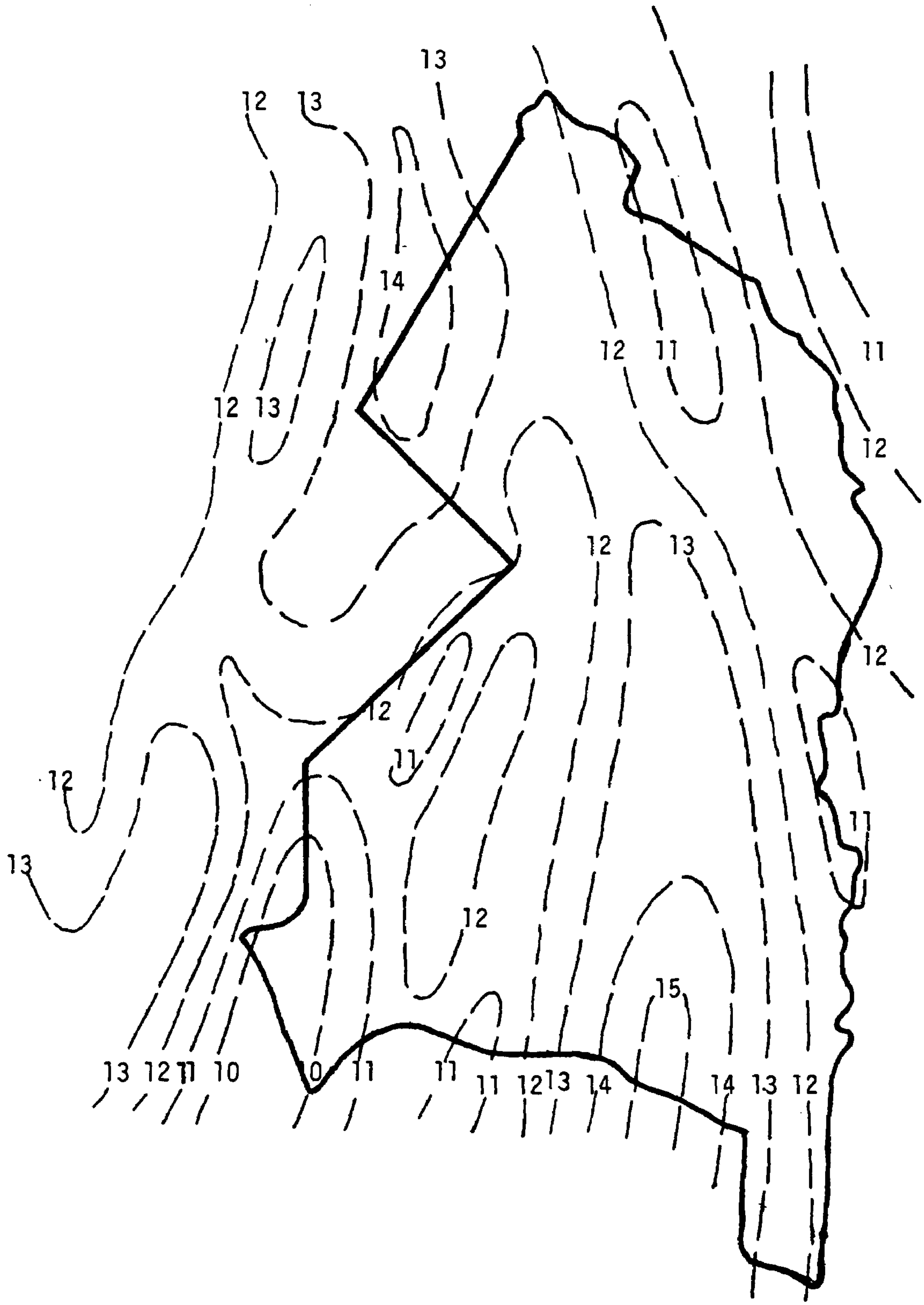

Figure 1. Monthly Precipitation (in.) over Prince George's County, Maryland in September 1975

tion, attaining a perfect match with recorded stream flow is nearly impossible. It should therefore be recognized that the discrepancies were not necessarily due to modeling errors. They could, in fact, have been due to errors in the recorded input data.

On this basis, the parameter adjustments for the hydrologic calibration were based on the results of the following analyses:

- Annual Water Balance
- Monthly Water Balance
- Mass Curves
- Comparison of Monthly Runoff
- Comparison of Daily Runoff Duration Curves
- Comparison of Hydrographs

The above methods were used to compare the simulated data with the recorded data.

ANNUAL WATER BALANCE

Following the efforts to characterize the accuracy of the precipitation and streamflow data, the next step in the calibration process was to adequately approximate the annual water balance. In this phase, the loss terms such as evapotranspiration and deep ground water, were adjusted so that the volume of discharge was approximately correct with no concern yet for timing.

Figure 2 shows the comparison of annual water yields during the calibration-verification period for the four cases at the Piscataway Creek USGS gauge. Generally speaking, the first few years had simulated annual runoff volumes higher than the recorded ones; while the simulated runoff volumes were lower than the recorded runoff volumes for the last few years. The next step of the annual water balance review required the preparation of mass curves. Mass curves provide a relatively straight-forward method for analyzing stream yield. While generally used for reservoir analyses, their use in this study was intended to graphically compare the annual variation between the simulated and historical data. Figure 3 shows the mass balance between the accumulated, simulated and recorded water yields. From the above two figures, it can be seen that including the effect of snow/ice accumulation and melting in a simulation study will certainly cause a better fit with the recorded data although this is not significant.

The major reason for discrepancies between recorded and simulated flows is that the actual storm patterns are often quite different from those used in the model. While the variations from storm to storm have a tendency to even out over time, Figure 1 shows that a spatial variation does exist. This variation could have a substantial effect on the annual water balance. Also, the land use conditions were assumed to be constant over the entire study period while in reality the conditions were constantly changing. To incorporate solutions to

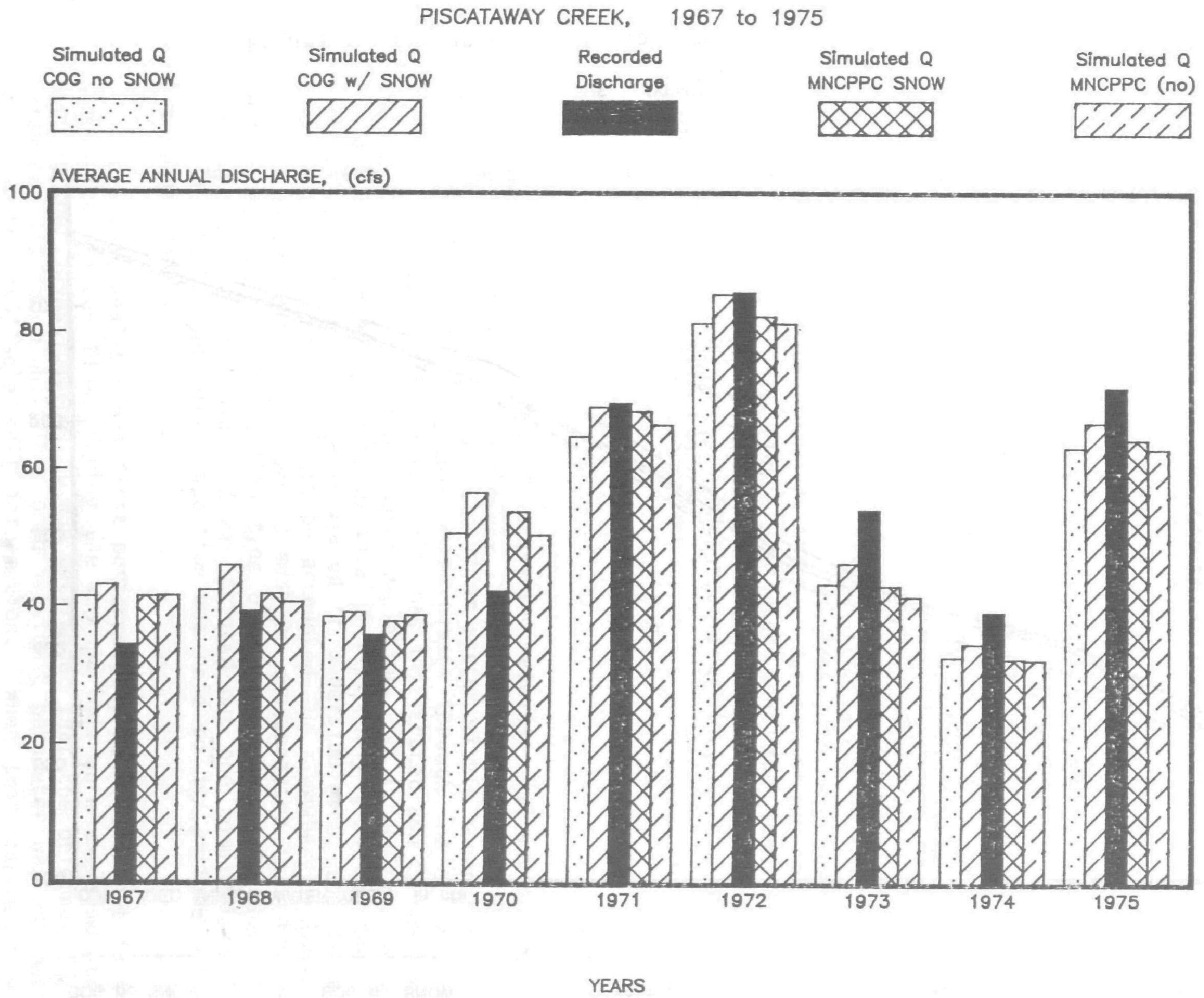

Figure 2. Comparison of Average Annual Discharges

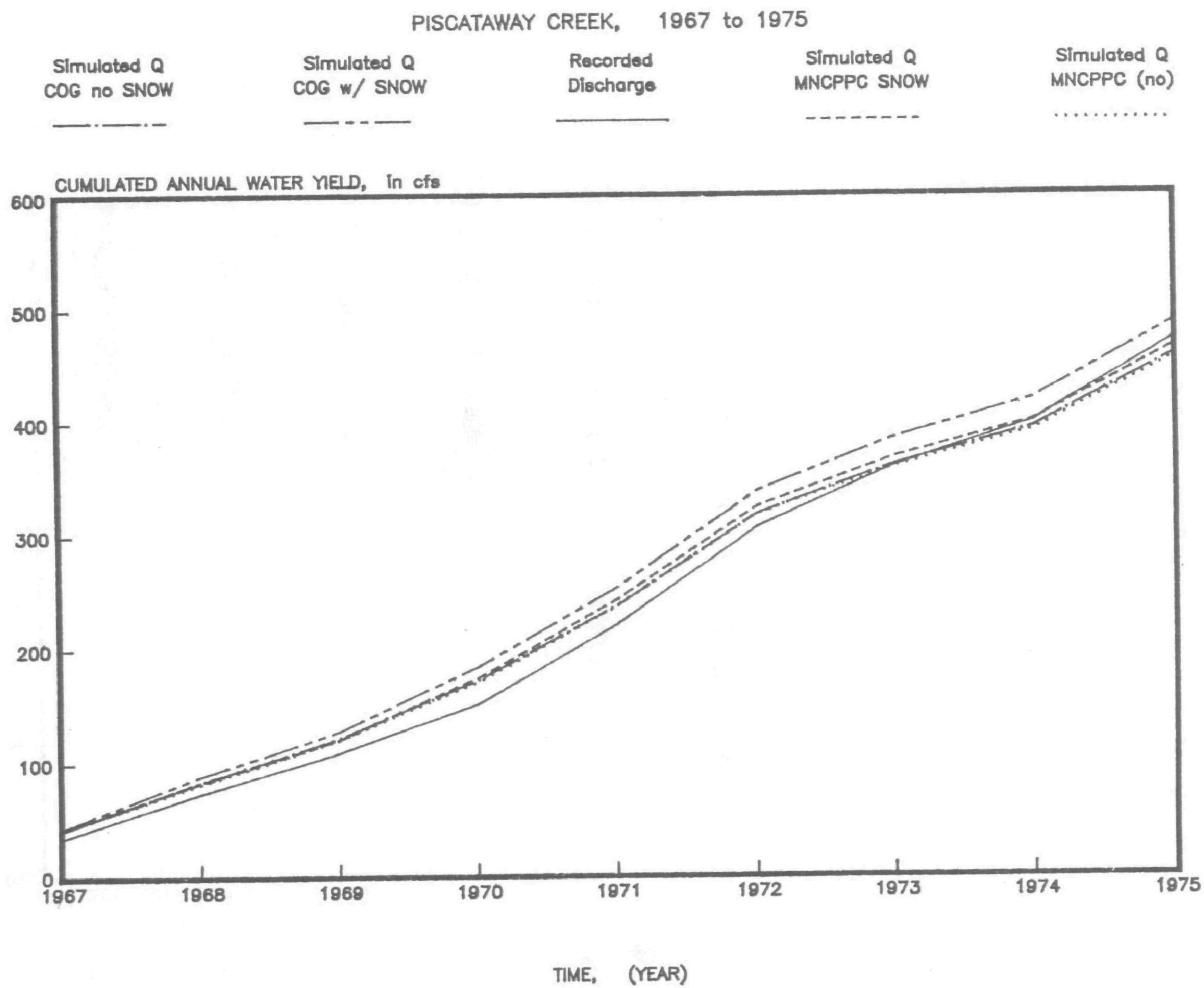

Figure 3. Comparison of Annual Mass Water Yields

these problems into the existing data base would have required an effort far in excess of the intended purpose. The current modeling effort wis not intended to perfectly match historical records, but rather, to produce a tool for use in evaluating environmental planning alternatives.

MONTHLY WATER YIELDS

The best method of evaluating the closeness of fit between simulated and recorded monthly discharge volumes is to present the data on a simulated vs. recorded plot. If the simulation is perfect, all of the plotted points would fall on the 45-degree line (1:1) indicating that each simulated monthly value was exactly equal to the recorded monthly value. The plot of monthly values of simulated and recorded runoff volumes for the four cases (conditions) is shown in Figure 4. The points tend to be randomly scattered and evenly distributed around the 45-degree line. The best fitting curve (straight line) for each of the four cases has been determined and also presented in Figure 4. This figure indicates that the simulated water balance on a monthly basis is quite representative. Also, this figure indicates that it is more representative to include the effects of snow/ice accumulation and melting.

CALIBRATION OF DAILY FLOWS

The rainfall event displays two results: (1) surface runoff and (2) groundwater replenishment. Surface runoff is a transient phenomenon and passes through the river system within several days of the rainfall event. Once this water passes through the system, the flow in the river is that water which has infiltrated and is now leaving the system through ground water depletion. It is obvious that if the annual and monthly water balance is reasonably good but the low flow periods are either oversimulated or undersimulated, the difference in the water balance must be accounted for somewhere. Thus, a few cfs errors over a dry period must be counteracted in a few wet weather events. To avoid this type of misrepresentation, the daily average discharge as simulated was printed out and compared to the recorded data. Particular attention was paid to dry periods during the summer months.

When this analysis was first performed, it was found that in the simulation, stream flows during the dry periods were oversimulated. Upon further investigation, it was determined that the original INFIL parameter was too high. This original INFIL parameter was estimated based on the available soil information. When this parameter was

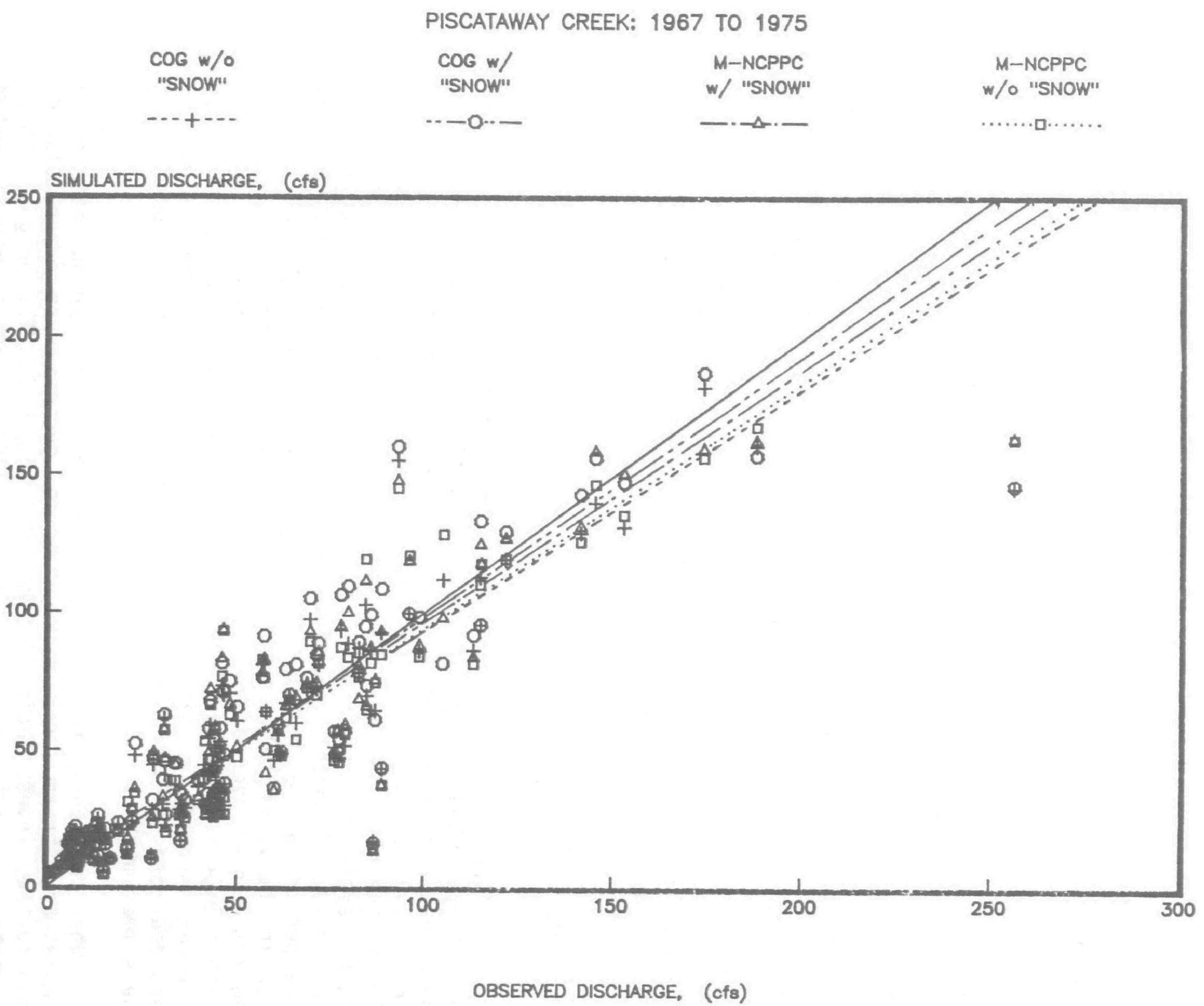

Figure 4. Simulated & Observed Monthly Water Yields

properly adjusted, the simulated daily flow duration curves matched the recorded data extremely well (see Figure 5). From this figure, it can be seen that the simulated daily flow duration curves for all four cases converge very well.

At this point in the calibration procedure, the simulated daily average discharge value matched the historical data well. Further refinement of the model calibration requires matching hydrographs. This process is critical for flood analyses.

CALIBRATION OF SIMULATED HYDROGRAPHS

When calibrating a continuous simulation model, it is rarely expected to match every hydrograph exactly. The real variation in the prototype varies far more than can be explained by any modeling effort. This variation is the major reason continuous simulation is far superior to single event simulation. When matching a hydrograph in a single event model, it is easy to justify increasing the antecedent soil moisture. This results in "turning up" hydrograph peak and hydrograph volume. Timing of the hydrograph can be adjusted easily by varying the time of concentration or any related parameters.

Continuous simulation, on the other hand, does not easily lend itself to such superficial calibration methods. Any assumption made for a given event must hold true for the entire period. Oftentimes, adjusting one parameter to fit a particular hydrograph would jeopardize the hydrograph shape of other storm events. Thus, a careful selection of the parameter values is extremely important in a continuous simulation model.

In the HSPF model, hydrograph shapes for selected storm events can be effectively altered with the UZSN (nominal upper zone soil moisture storage) and INTFW (interflow inflow parameter) parameters to better agree with observed values. Minor adjustments to the INFILT parameter can also be used to improve simulated hydrographs. Since adjustments to INFILT will change the established annual and monthly water balance, the changes to INFILT should, we felt, be somewhat minimal. Parameter adjustment was concluded when changes did not produce an overall improvement in the simulation.

The observed and four simulated hydrographs (one for each case) of the February 2, 1973 storm event are presented in Figure 6. This storm event was selected for not only is it a winter storm, it is also the maximum storm event for that year. From this figure, it is very evident that the consideration of snow/ice accumulation and melting processes is necessary. The peak flow is closer to the observed data

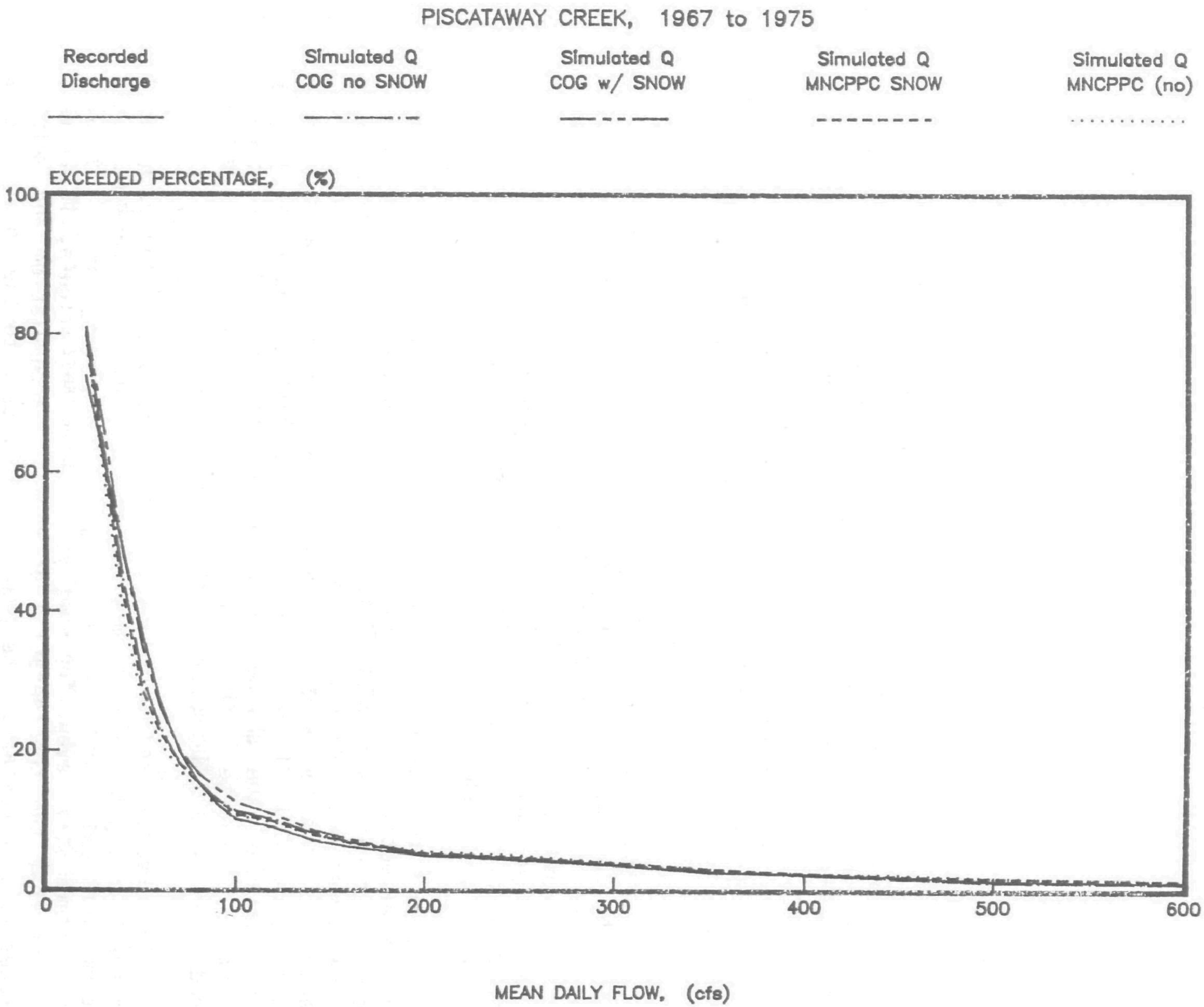

Figure 5. Comparison of Daily Flow Duration Curves

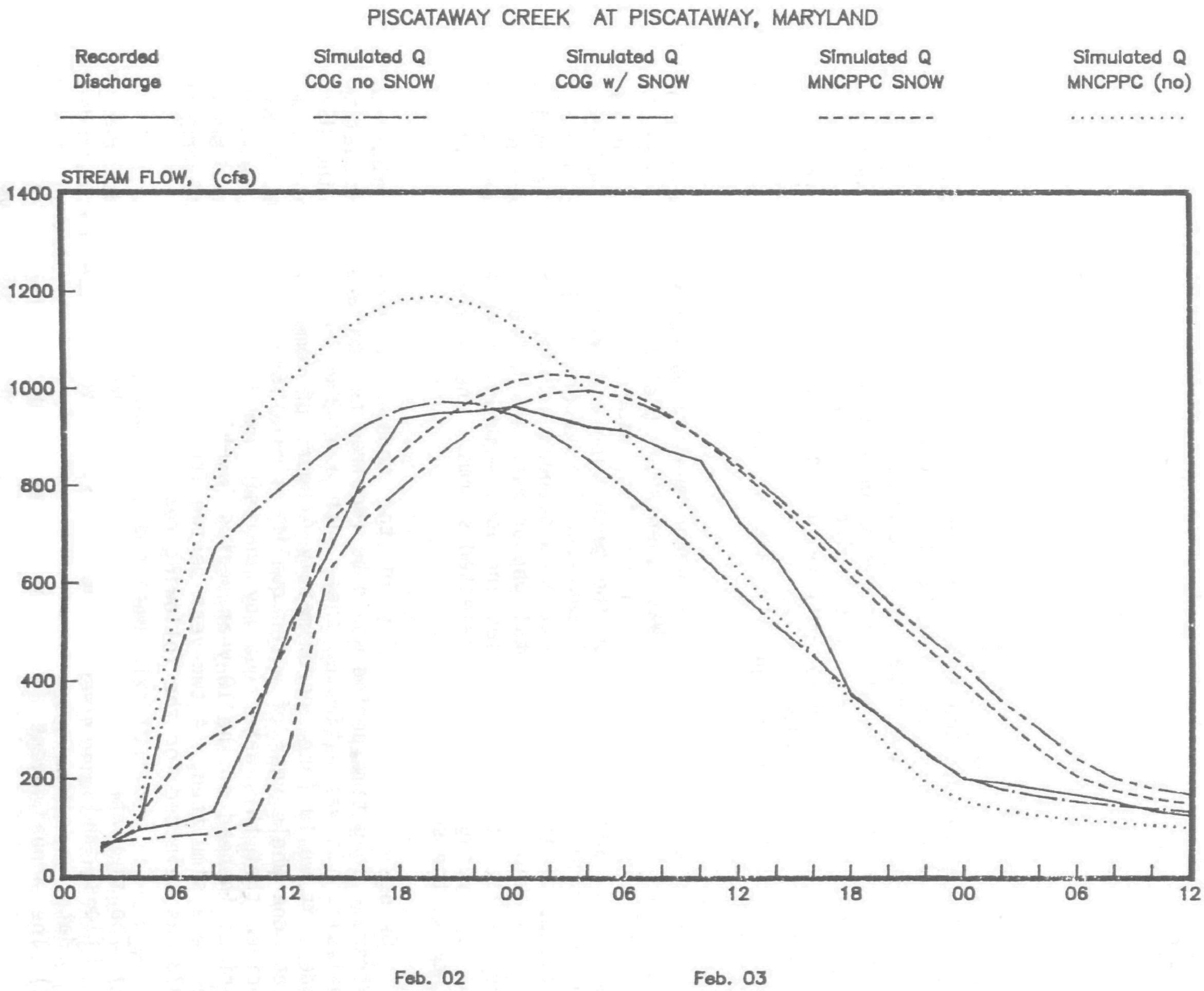

Figure 6. Comparison of 02/02/73 Storm Hydrographs

so also is the timing. It should be noted that the peak flow "time" for case 1 is almost identical to that for case 4. However, due to the use of a higher "upper zone storage" value in the winter period, the peak flow rate for case 1 is much smaller than the flow rate for case 4.

WATER QUALITY

The generation and transport of sediment and water quality constituents are very closely related to watershed hydrologic processes. Since all of the simulated annual water balance, monthly water yields, daily flows, and individual storm hydrograph appear to be very close to the recorded data, it is considered that the watershed model is very capable of simulating the water quality related responses of Piscataway Creek to all meteorologic conditions including winter periods. As the primary purpose of this study was to assess the effects of accumulation and melting of snow/ice on a watershed using the HSPF model, water quality and sediment calibrations were not performed.

In this study, the set of water quality parameters developed by COG in its Piscataway Creek Watershed study (Reference 1) was used for each of the input sequences of the four study cases to analyze the impact of "SNOW" addition on the generation and transport of sediment and nonpoint source constituents. The water quality constituents included in this study are total suspended solids, nitrate, ammonia, total organic nitrogen, total phosphorus, and BOD-5. Comparison of the results of the four cases includes monthly total loads and water quality hydrograph for two selected storms: one in the winter and the other in the summer.

Owing to the complexity of the model, it was felt that an extremely long time period would be required to simulate water quality analysis. It was estimated that, with a HP-3000 version of the HSPF model, it would take approximately 4 hours of computer time to complete one whole year of water quality simulation. Therefore, it was decided that this water quality analysis would cover only a two-year period; instead of the 10-year period in which the hydrologic processes were simulated. A two-year period from February 1971 to February 1973 was selected for the following reasons:

(1) The simulated annual water balance, monthly water yields, daily flows, and individual hydrographs are very close to recorded data;
(2) The annual event (with the highest peak flow) of 1972, which occurred from June 22 to June 23, is a representative summer storm; and
(3) The annual event of 1973, which occurred from February 2 to February 3, is a good winter storm.

MONTHLY TOTAL LOADS

Since the water quality processess are very closely related to stream flows, a comparison of simulated monthly average discharge among the four cases is presented for the same period in which water quality simulation was performed (see Figure 7). From this figure, the monthly average stream flows do not significantly change when snow/ice accumulation and melting are considered. Because of this insignificant difference in monthly stream flows when a completely identifical set of water quality related parameters is added to the HSPF input sequences, the monthly total loads of selected water quality constituents do not significantly change with the addition of the "SNOW" option. This finding is graphically presented in Figures 8 through 10. Monthly loads of BOD-5, total phosphorus, ammonia, and total organic nitrogen all showed very similar results.

INDIVIDUAL HYDROGRAPHS

As in the hydrologic process, one of the most important comparisons in water quality simulation is a comparison of individual hydrographs. Efforts are emphasized on the hydrographs of the largest storm event occurred in each year. In this case, the largest storm event for 1972 is the June 21 event, and the largest storm event for 1973 is the February 2 event. The former storm event is a typical summer storm event while the latter one is a winter snow storm event. The individual hydrographs for these two completely different types of storms were studied separately.

Figure 11 and Figure 12 present a comparison of bi-hourly hydrograph of total suspended solids and a comparison of bi-hourly hydrograph of nitrate, respectively, for the June 21, 1972 storm event. As expected, from these two figures, the time distributions of sediment loads and water quality were not significantly affected for a summer storm event when snow/ice accumulation and melting effects were included in the watershed model. The bi-hourly hydrographs of nitrate particularly were almost identical with and without the "SNOW" option. A similar situation has been observed for other water quality constituents as well.

On the other hand, however, the time distributions of sediment loads and water quality constituents would be quite different during a winter storm if snow/ice accumulation and melting effects are considered in the watershed model. The majority of water pollutants are not transported from the land surface in the early portion of a winter event. The pollutants are gradually conveyed from the land surface when the accumulated snow/ice starts to melt. In addition, with snow-covered land surface, soil detachment, washoff, and total sediment, are not as prodigous.

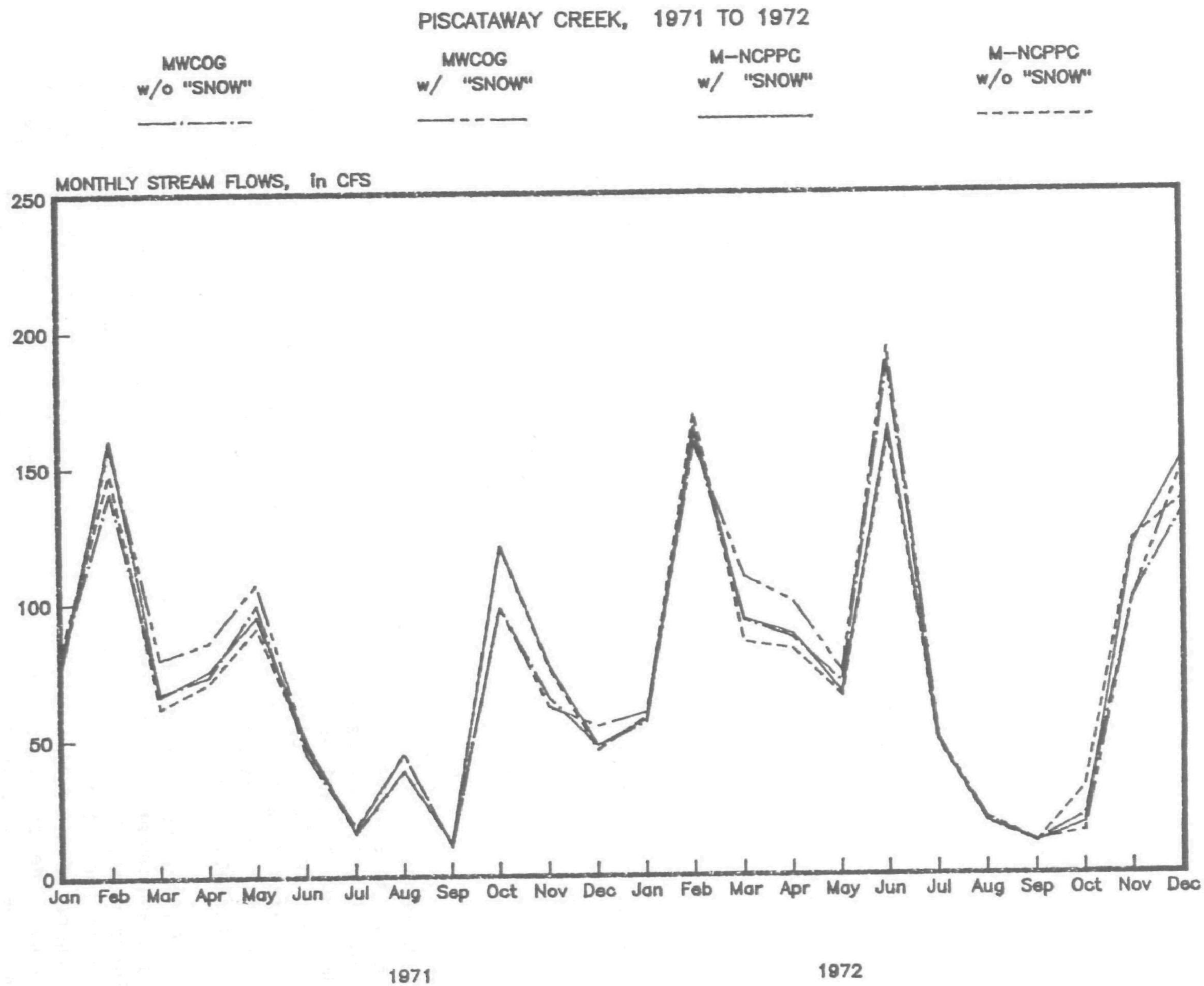

Figure 7. Monthly Average of Total Stream Flows

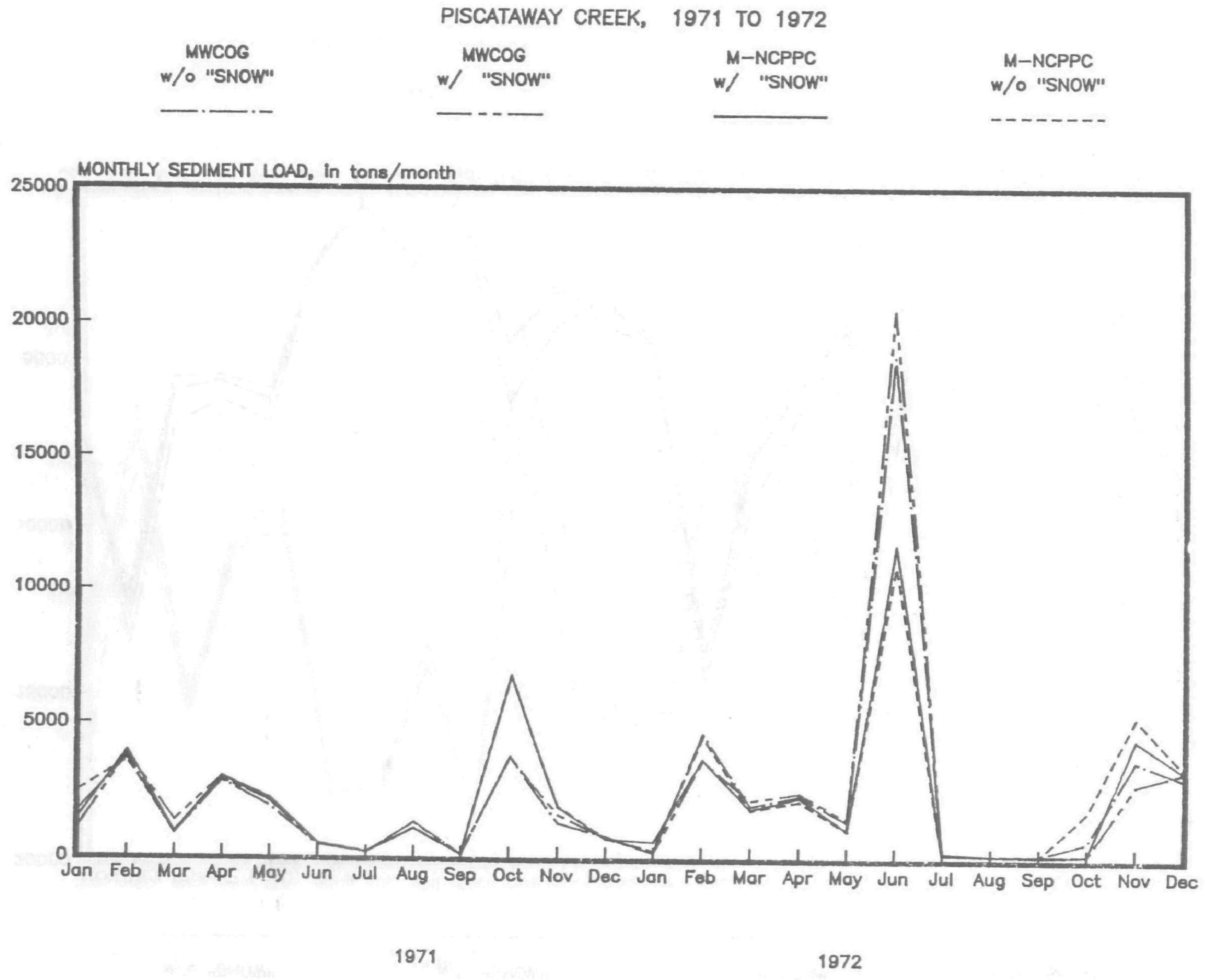

Figure 8. Monthly Mass of Total Suspended Solids

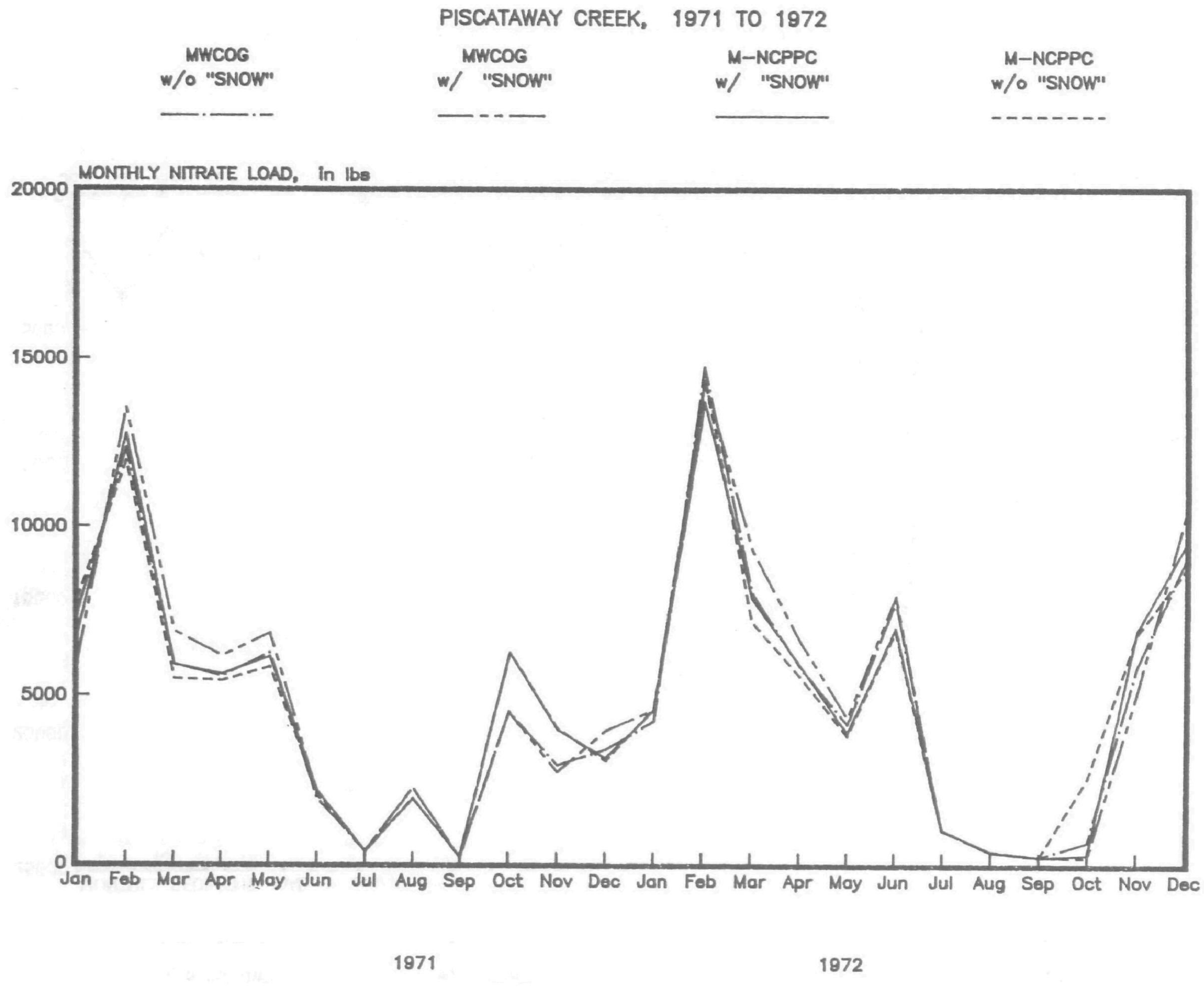

Figure 9. Monthly Mass of Nitrate (Load)

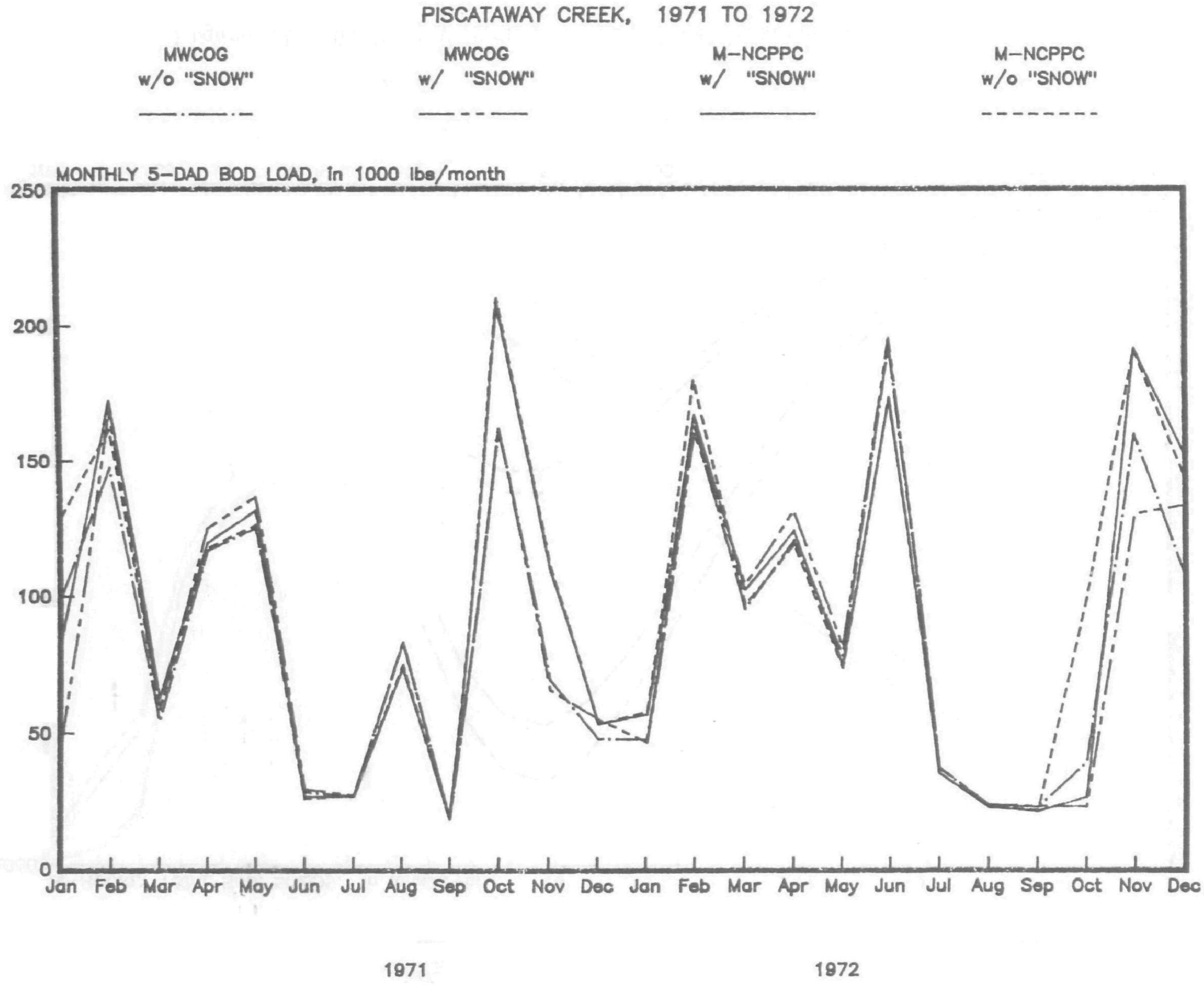

Figure 10. Monthly Mass of 5-Day BOD (Load)

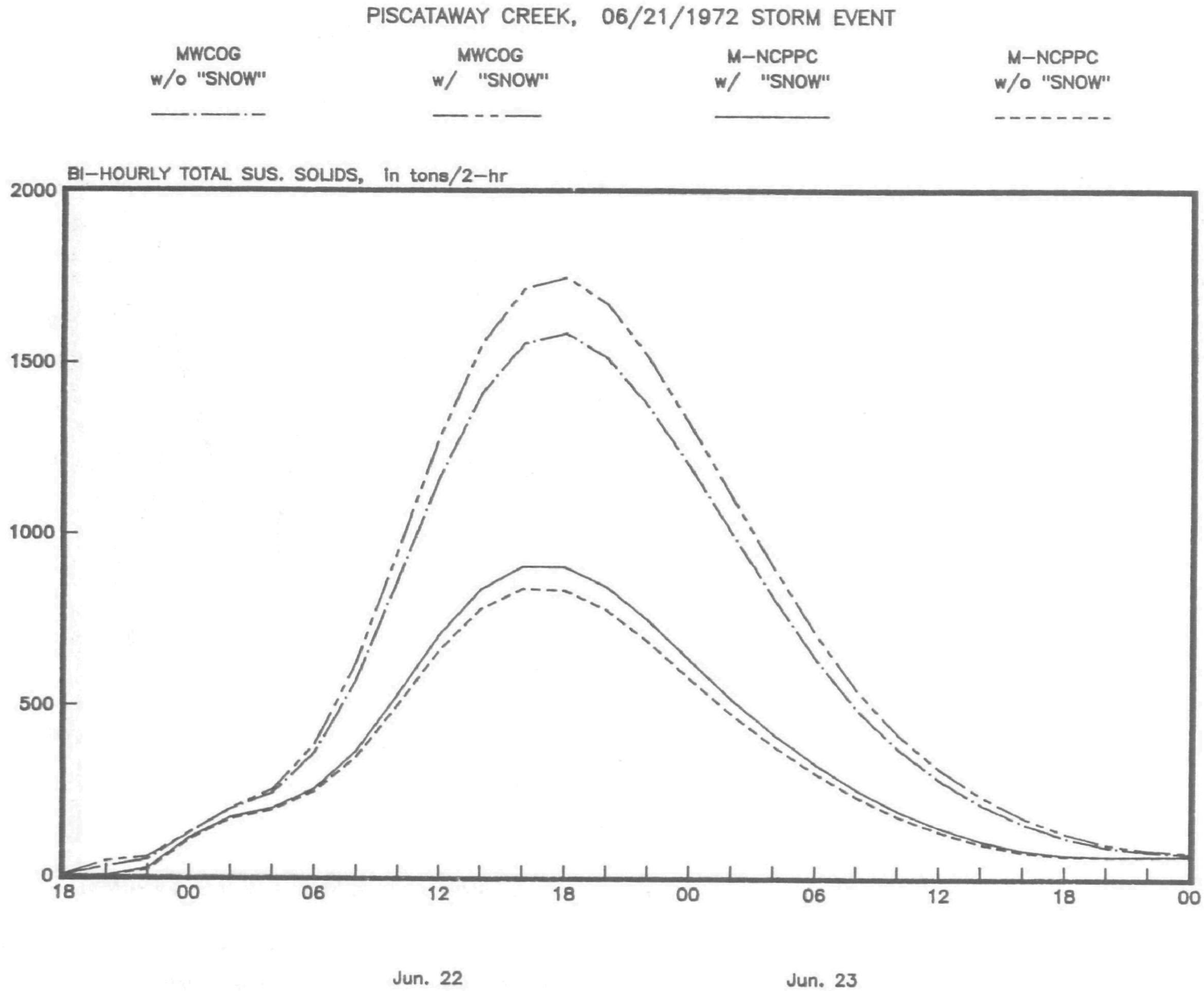

Figure 11. Bi-Hourly Hydrograph of Total Suspended Solids

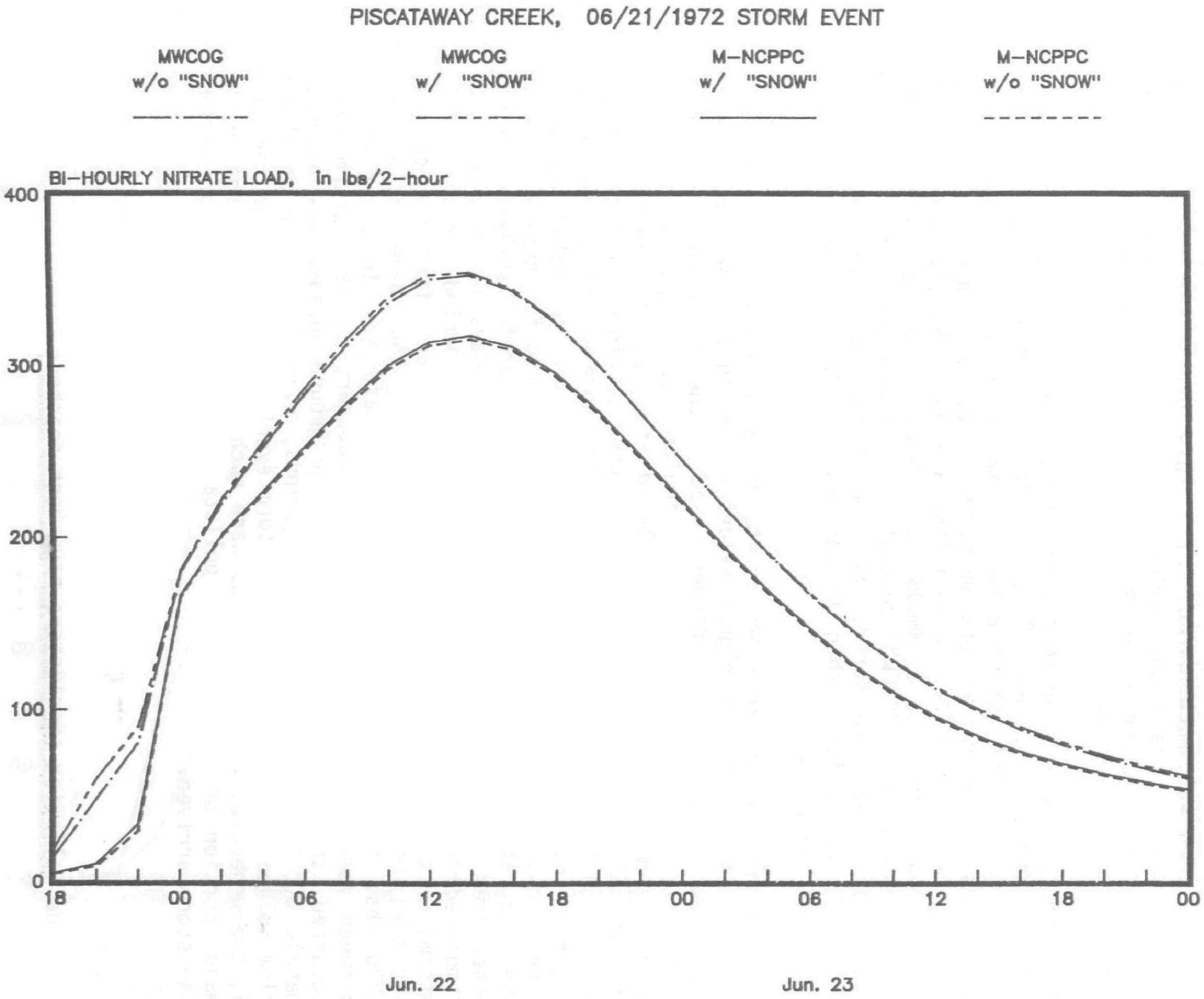

Figure 12. Bi-Hourly Hydrograph of Nitrate

Figure 13 graphically demonstrates the bi-hourly hydrographs of total suspended solids of the four different cases for the February 2, 1973 storm event. When snow/ice accumulation and melting effects were considered, the total suspended solids load was much less and the peak loading time was much later. For this particular event, "SNOW" effects reduced the total sediment yields by almost 50 percent, although the reduction of sediment load did not significantly decrease the total monthly load.

Figures 14, 15, and 16 present the bi-hourly hydrographs of total nitrate, phosphorus, and BOD-5, respectively, of the four cases for the February 2, 1973 storm event. From these three figures, shapes and trends of the hydrographs among the three water quality constituents appear very similar. Case 1 (MWCOG without "SNOW") generated the highest peak load, case 2 (MWCOG with "SNOW) was the next highest, and case 3 (M-NCPPC with "SNOW") generated the lowest peak load. When the "SNOW" option was considered, the time of the peak load was approximately 12 hours later than it was when "SNOW" was ignored in the watershed analysis.

On this particular storm event, it is of interest to note that because some relatively higher values of "monthly upper zone storage (MON-UZSN)" in winter periods were used in the COG study (case 1) to reflect the snow accumulation, the simulated peak flow for case 1 is much less than the peak flow simulated from case 4 (See Figure 6). Theoretically, higher flow rate should produce higher pollutant loads at a given time. In this particular event, however, it did not occur. Figure 14 through Figure 16 indicate that the generated pollutant rates for case 1 were larger than those generated for case 4 although the flow rate for case 1 is smaller. After a careful analysis, it was discovered that another storm had occurred 5 days previously. The simulated total flow volumes during that 5-day period (prior to this storm) were 1736 acre-ft for case 1 and 2001 acre-feet for case 4. And the simulated total nitrate loads during that same period were 2226 lbs. for case 1 and 3012 lbs. for case 4. In other words, between case 1 and case 4, when the February 2, 1973 storm event occurred, the available pollutant on the ground surface was approximately 800 lbs. less for case 4. Thus, even though the simulated flow volume during this February storm event was much higher for case 4, the generated pollutant loads were much smaller simply because the major portion of the available pollutants had been washed off before the storm arrived.

CONCLUSION AND SUMMARY

The following conclusions have been reached:

- The inclusion of "SNOW" option can only slightly improve the simulated annual water balance, monthly water yields and daily flows. However, it can significantly improve both shapes and

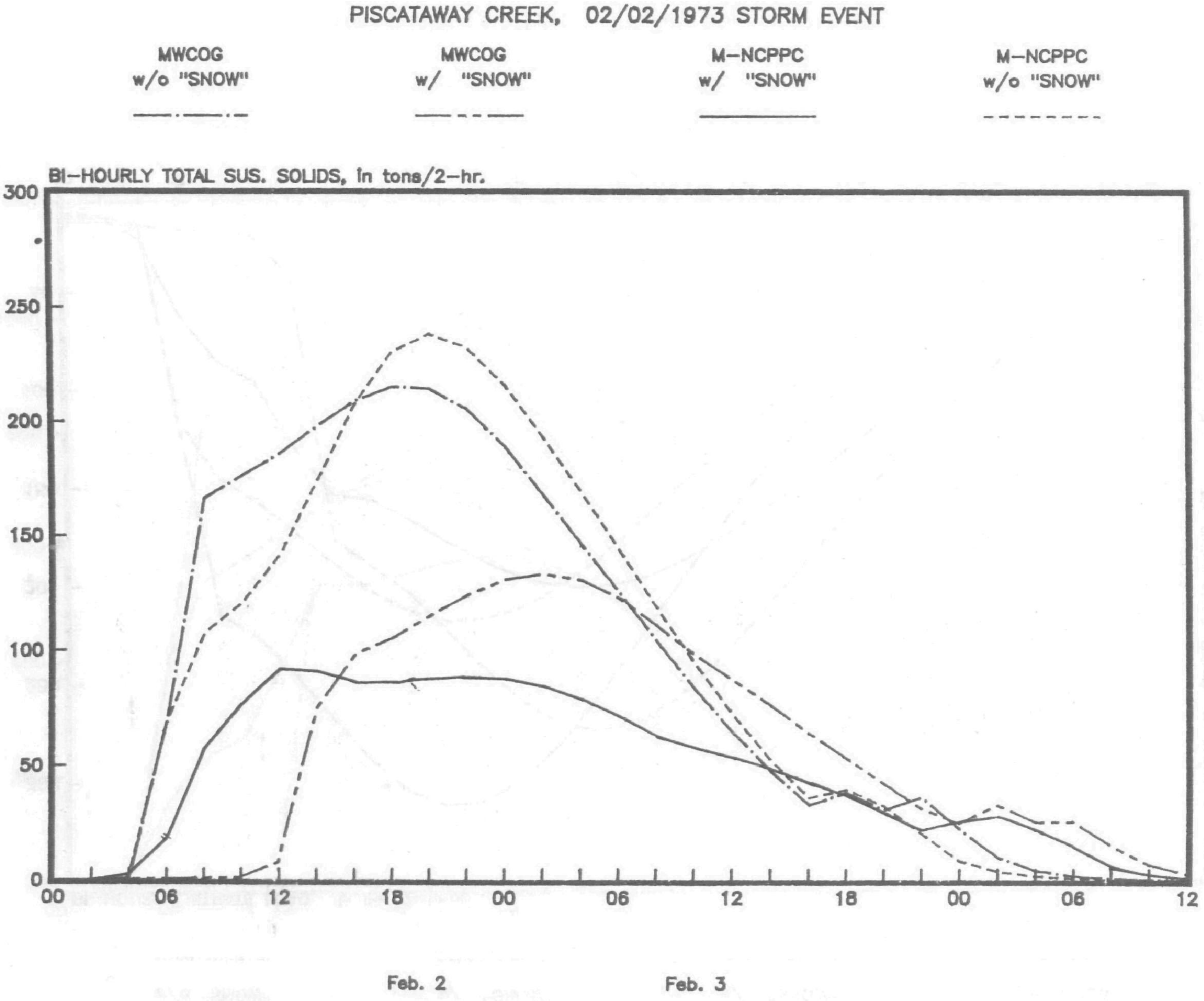

Figure 13. Bi-Hourly Hydrograph of Suspended Solids

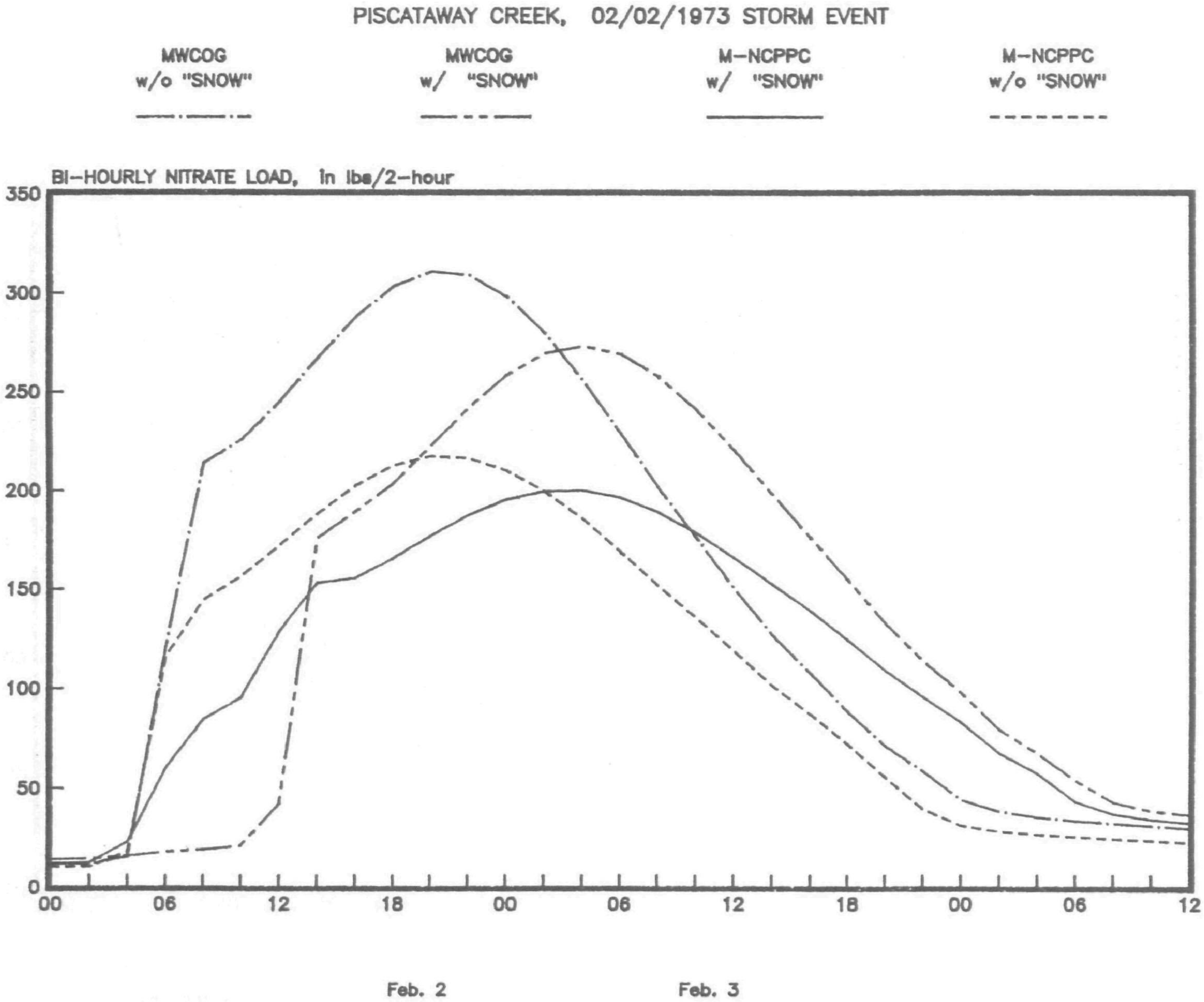

Figure 14. Bi-Hourly Hydrograph of Nitrate

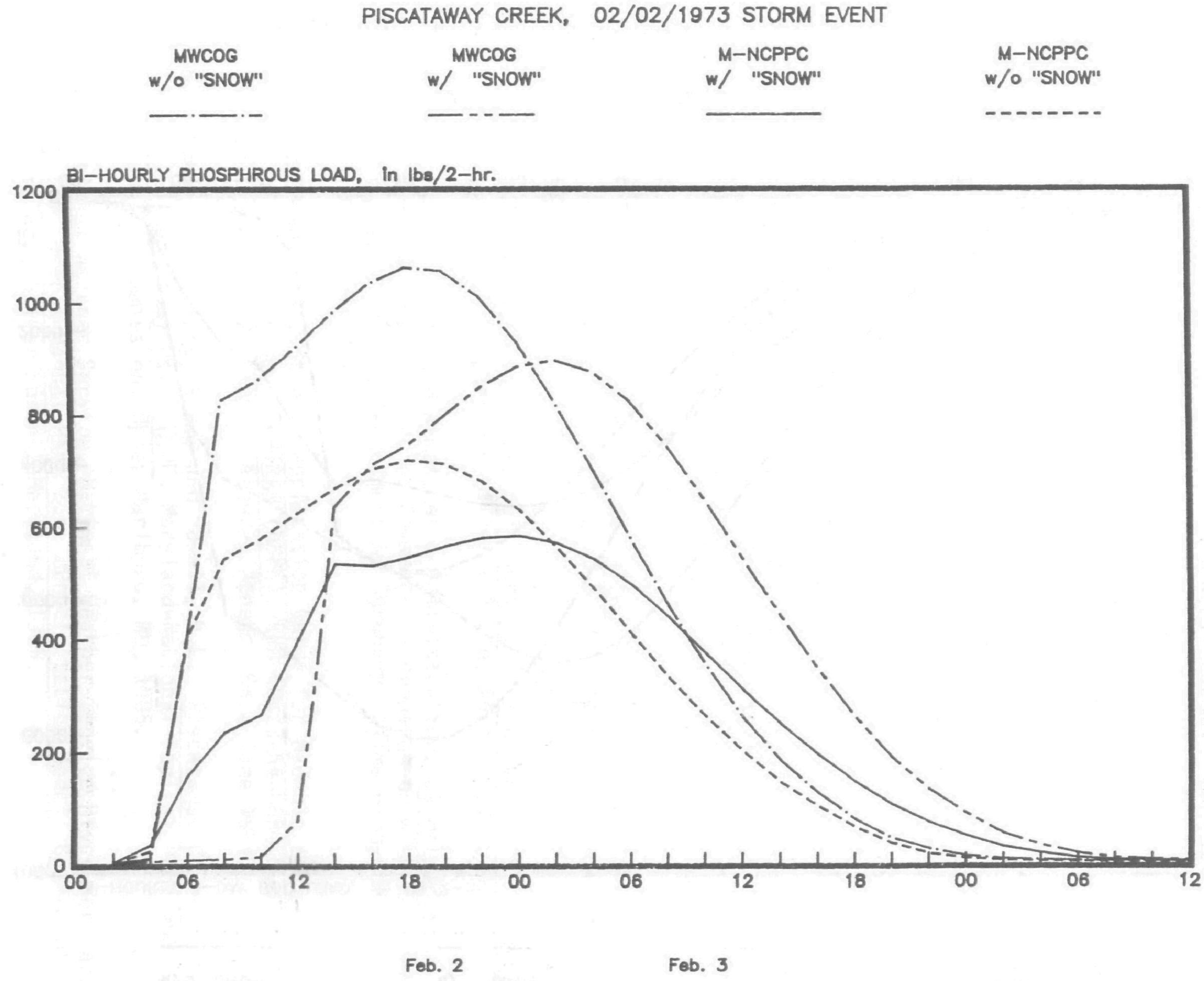

Figure 15. Bi-Hourly Hydrograph of Phosphorus

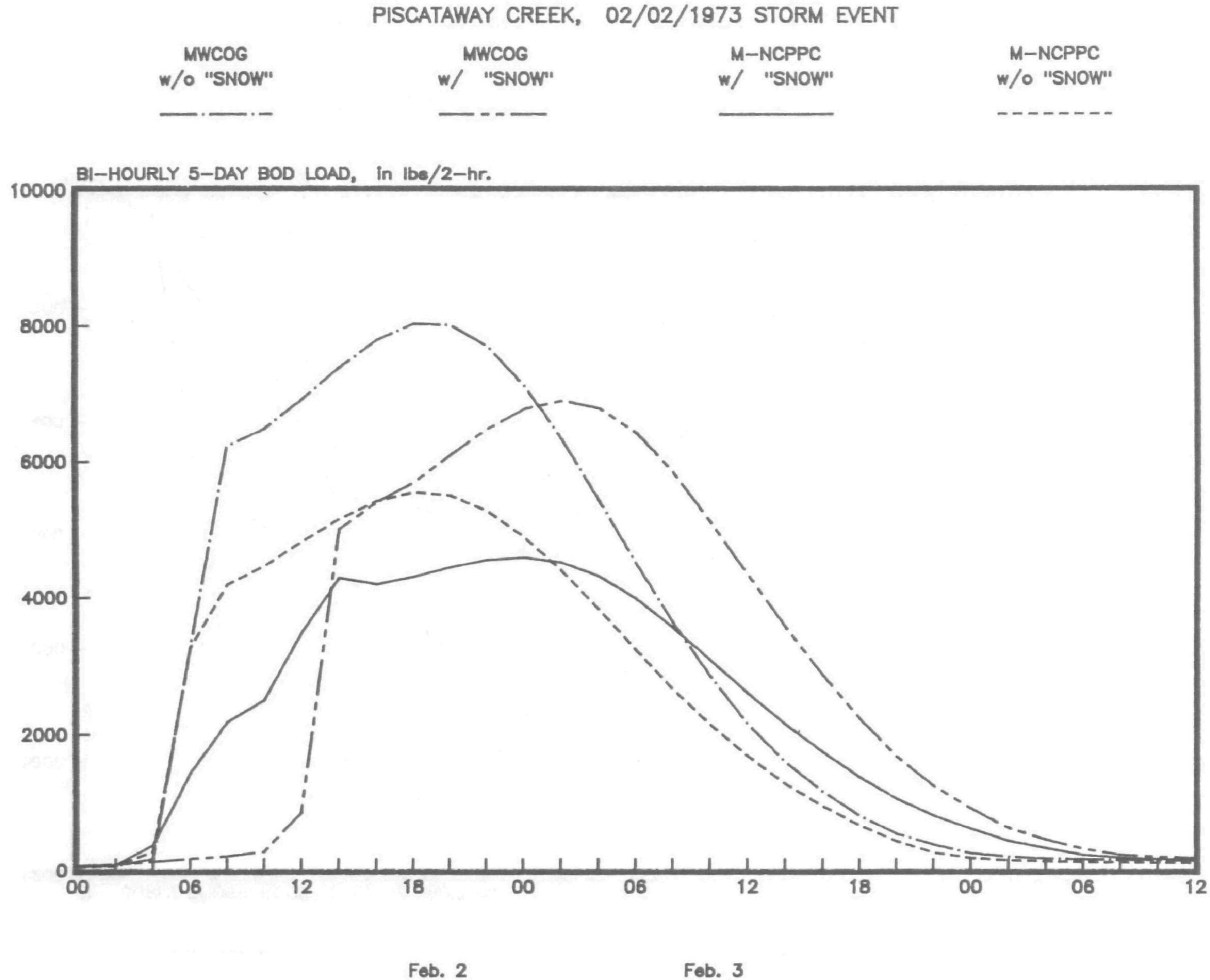

Figure 16. Bi-Hourly Hydrograph of 5-Day BOD Load

volumes of each individual flow hydrographs for those winter storms.

° Since the generation and transport of water quality constituents are highly dependent on the hydrologic process, the total annual and monthly pollutant loads will not be significantly altered when snow/ice accumulation and melting processes are ignored in the continuous watershed simulation study. But, if each individual pollutant hydrograph is the primary study goal, the "SNOW" option is certainly necessary.

° The HSPF model has been used successfully to simulate the hydrological response of Piscataway Creek. Furthermore, a good simulated hydrologic response tends to enhance a reasonable water quality simulation because of the close relationship between water quality and hydrologic processes.

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USE OF HSPF TO SIMULATE THE DYNAMICS OF PHOSPHORUS IN FLOODPLAIN WETLANDS OVER A WIDE RANGE OF HYDROLOGIC REGIMES

By

James C. Nichols, P.E., Staff Engineer
Michael P. Timpe, Senior Technician
Water and Air Research, Inc.
Gainesville, Florida

ABSTRACT

HSPF was utilized for the purpose of simulating the phosphorus dynamics within the floodplain wetlands of a major South Florida watershed in order to assess the water quality impacts of various proposed restoration alternatives. In order for the model to produce a realistic simulation, a number of modifications and enhancements were required within most of the program modules utilized. This paper will discuss the general approach taken to perform the water quality simulations as well as the modifications required to the HSPF source code in order to produce reasonable hydrologic, hydraulic and water quality simulations of the alternatives on a Harris 500 mainframe computer.

INTRODUCTION

A modified and extensively enhanced version of HSPF, release 7.0, was utilized to simulate the phosphorus dynamics within the floodplain wetlands of a major, channelized river in South Florida. The purpose of the study was to assess the potential water quality impacts of various restoration alternatives. The restoration alternatives under consideration incorporated a wide range of structural and nonstructural control techniques, including abandonment of the existing canal and attendent control structures, impoundment of major tributaries, recreation of floodplain wetlands by means of levees and control structures. An examination of baseline conditions and future no action conditions was also performed. Agricultural land use predominates within the watershed under study.

PROGRAM OVERVIEW

The HSPF program is a system of separate, task-oriented program modules designed to perform a continuous networked simulation of the hydrologic cycle and water quality behavior within a watershed and its receiving waters. The previous land segment module (PERLND) was used to simulate the upland and wetland subbasins and the RCHRES module was used to simulate the channel reaches. HSPF is a very large FORTRAN program written in a structured programming style. The program uses a large random access file to store intermediate results for subsequent input to the program when the entire simulation network could not be run in one job.

PROGRAM SELECTION CRITERIA

The HSPF program was selected for use after careful screening and comparison of the capabilities of existing water quality models in the public domain. Other models given serious consideration included:

- o Agricultural Runoff Management Model II (ARM-II),
- o Dynamic Estuary Model (DEM),
- o EXPLORE-I,
- o Nonpoint Source Model (NPS),
- o QUAL-II,
- o Stormwater Management Model (SWMM-III),
- o Storage, Treatment, Overflow and Runoff Model (STORM), and
- o Water Quality for River-Reservoir Systems (WQRRS).

Each model was subjected to a rigorous technical and applicability evaluation procedure. The technical evaluation considered the type of model, time variability involved, transport processes, water quality processes, constituents simulated, forcing processes, underlying assumptions and limitations of the model, and the data requirements. The model applicability evaluation considered the ease with which a given model could be applied, given the study objectives. An assessment was made of the availability of model source code, quality of model documentation and technical support, and the type and form of output generated by each program. Further consideration was given to computer hardware and software required for program implementation and the estimated run time and resulting computer costs.

HSPF was found to incorporate the following major advantages:

- o Use of a single model program package to perform the simulations for each of the major hydrologic components identified within the study area;
- o The program was capable of simulating, with modifications, the major features of each of the proposed restoration alternatives;
- o The water quality components of the model simulate clearly defined (although simplified) physical, chemical and/or biological processes;
- o Modern, topdown, modular program structure;
- o Output format flexibility and analysis capabilities;
- o Program developed and supported by the Environmental Protection Agency (EPA) Environmental Research Laboratory;
- o Flexible internal time steps;
- o Flexible time series data file manipulation capabilities; and
- o The utilization of a common data base structure.

In any modeling work there are inherent limitations and assumptions which may impact the usefulness of the results. These limitations include both structural limitations which result from initial model simplifications and/or assumptions which affect the ability of the model to accurately simulate reality and assumptions made relative to model input data that cause deviations from reality in the model results. HSPF has a number of significant structural limitations, not all of which were fully appreciated when the program was originally selected, including:

- o Momentum is not considered in the receiving water portion of the model, thus flows must be unidirectional and backwater effects are not taken into consideration;
- o Individual model elements are essentially one dimensional with only vertical variability taken into consideration;
- o Empirical hydrologic and nutrient transfer/transport process simulation algorithms greatly dependent upon the simulation time step;
- o Considerable program modification and enhancement was required to simulate hydrologic conditions found in wetland systems;
- o Considerable program modification and enhancement was required to simulate phosphorus processes occurring in wetland systems;
- o Cumbersome method to input external sources of water quality constituents into the model;
- o Lack of nutrient flow routing capabilities between certain program modules;
- o Dispersion is not taken into consideration,
- o Inability to simulate gradually varied reservoir stage fluctuation schedules;
- o Rigid program structure centered around a central common block cross linked to external data files;
- o High initial setup costs due to program complexity and use of non-standard FORTRAN extensions;
- o High computer run costs due to the large program overhead, highly segmented program structure coupled with mainframe virtual operating system characteristics, and the large number of data input/output operations.

HSPF PROGRAM MODULES UTILIZED

The PERLND Module of HSPF was utilized to simulate hydrologic and water quality behavior of the upland and wetland subbasins in the watershed. The following program sections were invoked: PWATER, to simulate major components of the water budget; PHOS, to simulate the phosphorus cycle and transport of phosphorus from the land surface; MSTLAY, to simulate soil moisture and fractional solute fluxes; and SEDMNT, to simulate erosion and transport of sediment and soil.

Section PWATER forms the key component of the PERLND module. The output time series generated by this subroutine are used as input by other PERLND module sections. The model performed a continuous simulation of the state of each of six moisture storage compartments within each subbasin: interception, surface detention, interflow, and upper, lower and active groundwater zones. A number of data sources were exploited to establish the analytical framework, including a computerized grid cell data base with a spatial resolution of 12.36 acres for fundamental framework information such as subbasin delineations, land use, topographic, vegetation, and soils association information; U.S.D.A. Soil Conservation Service (SCS) Soils Characterization Data for soils physical and hydrologic characteristics; and an extensive review of the literature.

Section PHOS was used to perform a continuous simulation of the state of four phosphorus storage compartments: plant, phosphate adsorbed to soil particles, detrital organic, and soil phosphate in solution. These phosphorus storages are simulated separately for each of four soil horizons, as well as soil solution phosphate phosphorus in the interflow compartment of each subbasin. Data sources exploited to establish the framework for this section included the Project GCDB; SCS Soils Characterization Data; and an extensive review of the literature to estimate soil adsorption isotherm coefficients as functions of soil properties, the seasonal magnitude of phosphorus standing crop for each major land use/vegetation group in the watershed, the magnitude and seasonal variation of the detrital phosphorus storage compartment for each land use/vegetation group, detrital mineralization and solution phosphate immobilization rates under oxidizing and reducing conditions, seasonal uptake rates of soil solution phosphate by viable plants, and seasonal plant dieback rates.

The RCHRES Module was utilized to simulate the hydrodynamics and water quality behavior of open water systems in the watershed. The following program sections were invoked: HYDR, to simulate the hydraulic behavior of receiving waters; ADCALC, to simulate longitudinal advection of dissolved and entrained constituents; and GQUAL, to simulate instream attenuation of phosphorus assuming first-order reaction kinetics.

Section HYDR forms the key component of the RCHRES module with the output time series generated by this subroutine used as input by other RCHRES module sections. The model utilizes the kinematic wave method to route streamflows and ignores momentum. In order to circumvent this limitation, a series of water surface profiles were developed using the HEC-2 program for each alternative. An analysis of historical streamflow records was performed to develop the expected range of streamflow conditions in the watershed. The results of these analyses were used to synthesize depth-discharge relationships for each model element within the floodplain, including open channel segments and wetland subbasins. In addition, these results were used to route flows between open channel RCHRES elements and floodplain wetland PERLND subbasins. Data sources exploited to establish the framework for this section included the cross sectional data to set up the HEC-2 simulations and a review of the literature.

SEGMENTATION OF THE WATERSHED

The study area is characterized by extremely flat slopes, seasonal rainfall, and long-term flooding. It lacks a well-developed dendritic drainage pattern and natural drainage pathways are typically poorly-defined. Soil moisture and surface depression storages and evapotranspiration processes dominate the hydrology of the watershed. Considerable man-made alterations include both modifications to the natural drainage system as well as a considerable development of entirely man-made drainage networks. The study area was conceptualized as consisting of three major hydrologic components:

- o An Upland, or Runoff Loading component;
- o A Wetland System component; and
- o A Receiving Water component.

HSPF has an internally dimensioned limit of 75 "operations" which can be performed during a simulation run. This limitation determines the total number of PERLND subbasins, RCHRES channel segments, DISPLY time series, etc., which can be incorporated into a given run. Therefore, the simulation runs for each of the alternatives were performed sequentially in a series of steps, with output from upstream simulations used as time series input to downstream model elements. The watersheds formed by existing canal control structures provided a logical breakpoint for the model. This allowed a reasonable spatial resolution to be utilized, a greater amount of output detail to be saved for subsequent analysis, and a shorter run time for a given simulation run.

UPLAND, OR RUNOFF LOADING COMPONENTS

The Runoff Loading components were simulated utilizing the PERLND Module of HSPF. These watersheds were simulated independently of the Wetland and Receiving Water Model components with the output time series generated by the simulated watersheds used as input to the appropriate downstream elements. This approach was used since differences between alternatives only occur in the wetlands and receiving waters of the watershed. Since HSPF is hydraulically unidirectional, any backwater affects cannot be simulated. The purpose of the Runoff Loading Model component was to simulate the hydrology and processes affecting the transport of phosphorus from the land surface in the "upland" portions of the watershed. In this study, the upland portions of the watershed were considered to be those areas outside of the floodplain of the river and major tributaries, and/or upstream of areas directly impacted by existing or proposed structural controls.

A total of 22 upland runoff loading subbasins were employed to perform the water quality simulations for all of the alternatives. Since none of the alternatives simulated directly impact the hydrology or land use projected for the upland subbasins, the runoff loading model simulations were performed one time for each land use scenario for the projected impact scenarios. The resulting output time series from each of these simulations were then saved and utilized as input time series to the wetland/receiving water model for the appropriate impact scenario. Upland subbasin delineation, for the most part, was determined by the delineation of subbasins within the floodplain. Where feasible, a separate upland subbasin was delineated for each wetland subbasin. Tandem upland subbasins, i.e., subbasins hydrologically in series, were not utilized.

WETLAND SYSTEM COMPONENTS

The purpose of the Wetland System Model component was to simulate the hydrology and processes affecting the transport, retention, and release of phosphorus through the wetland systems impacted or created by the various alternatives. These systems included such features as tributary impoundments, impounded wetlands, flow-thru marshes, and the floodplain wetlands under various hydrologic regimes. Since most of the alternatives simulated in this study were designed to impact the hydrology of the floodplain wetlands, the establishment of the framework for the Wetland System Model elements received a disproportionate share of attention. In this study, the wetland component of the watershed was considered to consist of the areas within the floodplain and/or major tributaries. Between 23 and 26 wetland subbasins were employed to perform the water quality simulations for the various alternatives.

The extensively modified and upgraded version of the PERLND Module of the HSPF Program was utilized to simulate hydrologic and water quality behavior of the wetland system subbasins. The simulation of the hydrologic behavior and transport of water from the land surface was performed by Section PWATER. The primary difference between the upland and wetland subbasins was the algorithm used to simulate surface hydrology. The original algorithm treating overland flow as a turbulent process was used to simulate direct surface runoff in the upland subbasins. A new algorithm employing a functional depth-discharge relationship was utilized to simulate direct surface runoff from the wetland subbasins.

RECEIVING WATER COMPONENTS

The major open surface water components of the watershed were simulated utilizing the RCHRES Module of HSPF. The Wetland System and Receiving Water components of each alternative were simulated simultaneously. In this study the receiving water portions of the watershed were confined to open channels as well as any operational control structures located on these waterways. The Receiving Water component incorporated between 15 and 18 separate reach segments to simulate the hydrodynamic and phosphorus transport processes.

No mechanism was available in HSPF to simulate a continuously varying upstream pool elevation, in either the floodplain wetlands or in the receiving waters of C-38, as proposed in a number of alternatives. Therefore, a feature was added to the HSPF to approximate this operation in a step-wise fashion on a monthly basis. The operating rule was assumed to be applied in discrete, 0.5-foot monthly increments.

MODEL NETWORKING AND SIMULATION LOGISTICS

The pathways by which the above model components were connected were made by means of the NETWORK Block, between upstream and downstream model elements within a given model run, and/or by means of the EXT SOURCES and EXT TARGETS Blocks, between upstream and downstream elements simulated during separate runs. Figure 1 illustrates the physical arrangement of the model elements within a major segment of the system modeled and a schematic of the networked model elements utilized to perform the simulations.

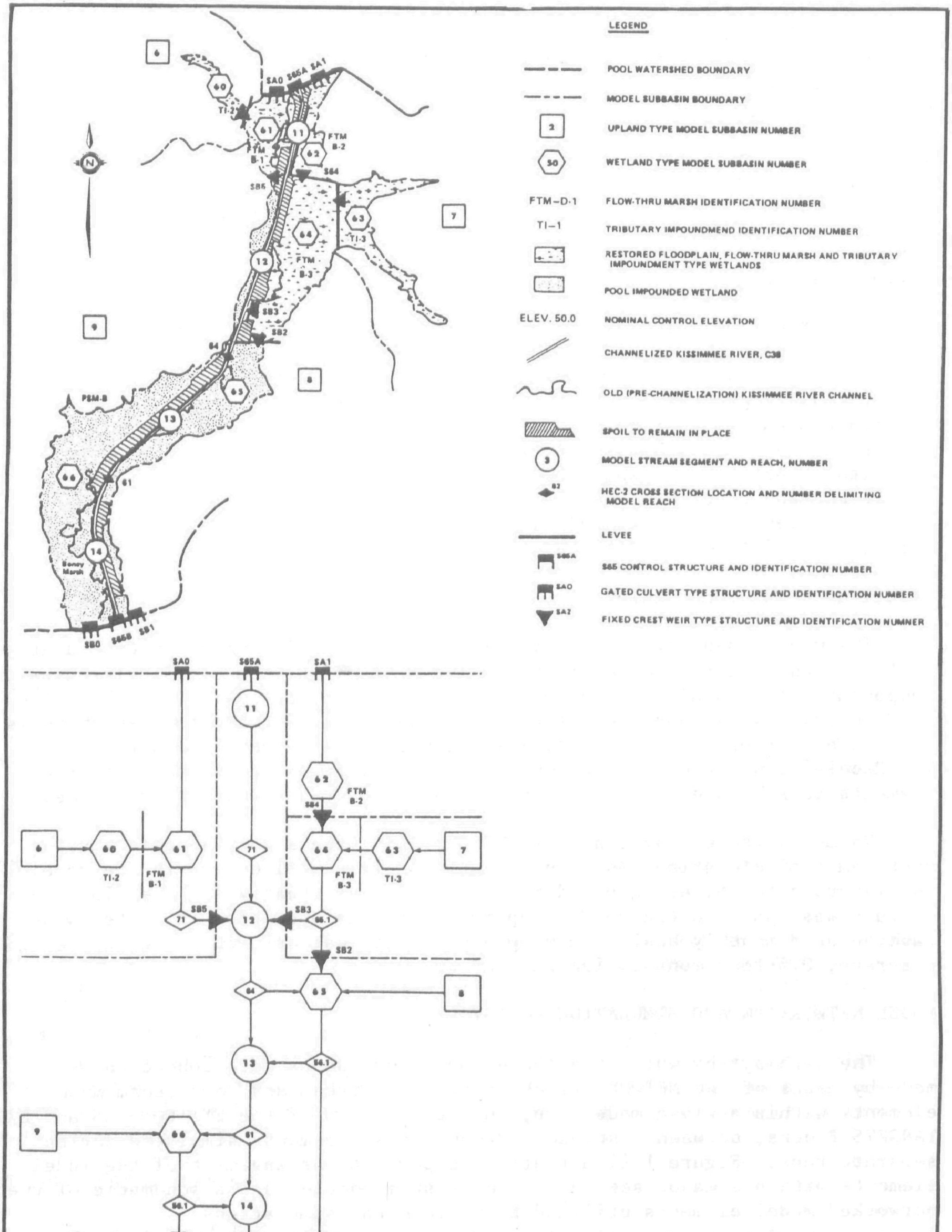
LEGEND
POOL WATERSHED BOUNDARY
MODEL SUBBASIN BOUNDARY
UPLAND TYPE MODEL SUBBASIN NUMBER
WETLAND TYPE MODEL SUBBASIN NUMBER
FTM–D-1
FLOW-THRU MARSH IDENTIFICATION NUMBER
TI–1
TRIBUTARY IMPOUNDMEND IDENTIFICATION NUMBER
RESTORED FLOODPLAIN, FLOW-THRU MARSH AND TRIBUTARY IMPOUNDMENT TYPE WETLANDS
POOL IMPOUNDED WETLAND
ELEV. 50.0
NOMINAL CONTROL ELEVATION
CHANNELIZED KISSIMMEE RIVER, C38
OLD (PRE-CHANNELIZATION) KISSIMMEE RIVER CHANNEL
SPOIL TO REMAIN IN PLACE
MODEL STREAM SEGMENT AND REACH, NUMBER
HEC-2 CROSS SECTION LOCATION AND NUMBER DELIMITING MODEL REACH
LEVEE
S65 CONTROL STRUCTURE AND IDENTIFICATION NUMBER
GATED CULVERT TYPE STRUCTURE AND IDENTIFICATION NUMBER
FIXED CREST WEIR TYPE STRUCTURE AND IDENTIFICATION NUMNER
SA0
S65A
SA1
TI-2
FTM B-1
FTM B-2
SB5
SB4
TI-3
FTM B-3
SB3
SB2
PSM-8
Boney Marsh
SB0
S65B
SB1

FIGURE 1.

First, the simulation runs for the Upland/Runoff Loading Model subbasins were performed. The times series representing the predicted outflows of water and phosphorus fluxes were saved in the proper Time Series Store for subsequent input into the appropriated downstream model segments. The model simulations for each alternative were then performed, proceeding from upstream to downstream. The predicted model outputs of discharge and phosphorus fluxes from upstream model elements were used as inputs, and the predicted outflows of water and phosphorus fluxes were saved in the proper Time Series Store for subsequent input into the appropriate downstream model segments or impact analysis.

MODEL TEMPORAL AND SPATIAL RESOLUTION

MODELING TEMPORAL RESOLUTION

A total period of 13 months was employed for all simulations. The calendar time employed for each simulation run was from September 1 through September 30 of the following year. The first month of the simulation was used to stabilize the hydrology and water quality of the model. This served to reduce the impact of errors in estimating initial conditions. The model predictions for the 1-month "warm-up" period were not taken into consideration in the subsequent evaluation of alternatives. The simulation period from October 1 through September 30 of the following calendar year, used in the evaluation of alternative impacts, was selected to correspond to the standard water year, which begins roughly at the end of the wet season. The externally input time series, employed as model forcing functions, were identical during the first and last month. Simulated output time series, used as input into downstream model elements, were used directly for the entire 13-month simulation period.

MODEL SPATIAL RESOLUTION

The Project GCDB provided the data base used to establish the framework for the PERLND subbasins of the water quality model. The spatial resolution of the model was determined by the 12.36-acre size of the individual grid cells. Similarly, subbasin; location; existing and future land use; vegetation groups; topographic data; and soil association related data affecting the development of the various hydrologic model parameters as well as the model parameters used to simulate the application of phosphorus to the land, the cycling of phosphorus within a watershed, and the transport of phosphorus from a watershed were developed around the information contained within the project GCDB.

HSPF is capable of simulating each of four soil horizons, the surface layer upper zone, lower zone, and active groundwater storage. The soils association characteristics were compiled into four horizons. Based on an analysis of the available soils data, previous modeling conventions, and site-specific conditions, the surface layer was taken as the 0-10 centimeter horizon, the uper zone as the 10-50 centimeter horizon, the lower zone as the 50-150 centimeter horizon, and the active groundwater storage to be the 150-180 centimeter horizon. Average soils characteristics for each soils association were calculated using a weighted average of the corresponding characteristics for component soil series. Weighting factors were based on the estimated percent coverage of each soil series within the association and a depth weighting factor based upon the proportion contained within a given horizon. Composite soil parameters for each subbasin were generally determined by calculating a weighted average of soil association parameters, based on the number of grid cells in the PERLND assigned to each association.

MODEL FORCING FUNCTIONS

Externally input data used to drive the dynamics of the simulations included precipitation, evaporation, air temperature, outflow and phosphorus loading from the upper watershed, as well as diffuse inputs of phosphorus to the watershed.

A total of 13 daily precipitation input datasets were synthesized using statistical modeling techniques and a 21-year historical rainfall record for nine National Oceanic and Atmospheric Administration stations surrounding the watershed. Average daily pan evaporation and air temperature data for the watershed were also estimated using data from surrounding NOAA stations. Inflows from the upper watershed were estimated based on a statistical analysis of streamflow records at the outflow from the upper watershed, prior and subsequent to the installation of controls. Phosphorus loadings from the upper watershed were estimated based on an analysis of historical water quality at the same location. Diffuse, external inputs of phosphorus to the watershed were estimated for present and future conditions based on an analysis of bulk precipitation data and supplemental cattle feeding and fertilizer application practices in the watershed.

PROGRAM MODIFICATIONS

A number of modifications were made to the HSPF program to improve its simulation capabilities for utilization in wetland systems. In addition, a number of programming errors; model deficiencies encountered as the modeling work proceeded; and hardware, operating system, and compiler related inconsistencies were identified and corrected. Model enhancements, corrections, and/or modifications made to the hydrologic algorithms included:

- A revised flow routing algorithm employing a depth-discharge, functional relationship from up to two possible exits;
- An enhancement to the revised flow routing algorithm to allow independent control of outflow on a monthly basis;
- The incorporation of a surface outflow time series corresponding to the second possible outflow exit to allow complex networking schemes between model components to be implemented;
- Modifications to the infiltration/percolation algorithms to improve simulation capabilities for saturated soil conditions.

Enhancements, corrections, and/or modifications made to the water quality algorithms included:

- A phosphorus "dieback" loop coupling the plant phosphorus compartment to the detrital phosphorus compartment with a time variate process function;

- o The algorithm used to simulate uptake of plant phosphorous from the available solution phosphorous storage was modified to depend on the plant phosphorous storage;
- o The capability to simulate the upward transport of nutrients between soil horizons corresponding to the nutrient pumping action of plants and to vary the ratio of aboveground to belowground plant phosphorus storage on a monthly basis;
- o Revision of the algorithm simulating detrital organic phosphorus mineralization in order to take into account reducing conditions typically found in flooded wetlands soils.
- o An improved mechanism for the input of external nutrient loadings and the incorporation of solution phosphorous inflow time series corresponding to the hydrologic flows to provide nutrient flow routing capabilities lacking in the original program.

The surface layer input was made to serve as the input point for all external nutrient loadings, such as from adjacent subbasin inflows, adjacent channel overbank inflows, atmospheric fallout, fertilizer inputs, etc. As a result of the sequence in which calculations are performed by HSPF, the phosphorous loadings to the surface layer were added directly to the storage compartment of phosphate adsorbed to the soil in the surface layer.

Fortunately, many localized algorithm changes in the program were fairly easy to implement on the Harris system and were thus readily attempted.

PROGRAMMING STRATEGY AND CONSTRAINTS

The initial intention was to make modifications to the program in conformance with the original structure of the system and with full retention of existing capabilities. However, this approach could not be followed because of the number and extent of program revisions which would have been required. The main COMMON block used by HSPF is employed in literally hundreds of locations throughout the program and is intricately cross-linked to the external library file INFOFL. This file is also internally cross-linked. The cross-linking is so extensive that major program modifications would have been required to incorporate even minor changes affecting this underlying structure.

As a result, modifications to the program made use of unused portions of the source code, such as existing input tables and time series connections, wherever possible. A number of new arrays were created in the limited free space available in the main COMMON block or in the space formerly allocated to inactive sections. Available time series and COMMON block space from section SNOW of the PERLND module were put to new uses, effectively deactivating that section. The land use block capability of the PWATER section was found to be of little practical use in its current form. Several time series originally used by this option were utilized for other purposes in the modified program. A number of PWATER and PHOS input data tables were used to provide data needed by the modified algorithms, effectively eliminating the unused original options. Non-functional changes pertaining to inactive sections and options were made as required to satisfy compiler syntax constraints and to maintain the required program structure framework. Fortunately, a number of the required algorithm enhancements had only local effect in the program and could be designed to work around the limiting structural elements.

HYDROLOGIC SIMULATION ENHANCEMENTS

The original PWATER section did not have the capability to adequately simulate the controlled surface hydrology which will occur in the various impounded wetland systems incorporated into a number of the alternatives. As a result, the use of function tables supplying stage-discharge relationships, "PTABLES", was implemented. Difficulties encountered in optimizing a networking scheme indicated that a second outflow, also under PTABLE control, was needed in the model. This allowed wetland systems with multiple outlet control structures to be simulated, and the capability of routing floodplain wetland outflows independently to both downstream reach and/or wetland elements.

The PTABLE routing algorithm was developed for the PERLND module similar in concpet to the FTABLE system used in the RCHRES module. A functional table of surface water depth values and hourly outflow rates are supplied to control the outflow from each of two outflow points. Multiple columns of outflow rate data may be included, to be selected for use on a monthly basis. This provides for variations in the control regime of impounded wetlands as well as for the seasonal fluctuations of naturally flooded wetlands and allows more complex networking schemes to be implemented. The upland subbasins used the original routing algorithm. A time borrowed series, SUROB, was required for the newly created second outflow gate. This provided the networking capabilities required to simulate the complex floodplain systems being simulated.

The modifications outlined here required program changes in the input section PPWATR, addition of the input subroutine PTABLE, changes in the calculation routines of Sections PWATER, SURFAC, DISPOS and PROUTE, and updates to the independently installed INFOFL used by HSPF. The PTABLE data was input to the program in the same format as an FTABLE. The first column corresponding to depth, in inches, the remaining columns corresponding to outflow rates, in inches per hour. Input table type MON-INTERCEP provided the monthly outflow column selector for use with both possible outflow PTABLES. The FTABLE/PTABLE to be used by the first and second exits was specified in the input table PTABLE-PARMS. A value of zero was used for this parameter to deactivate the pertinent calculations if a second outflow exit was not utilized. The modified time series SUROB was used to represent the outflow time series corresponding to the second outflow exit.

In the course of model calibration, further hydrologic modifications were found to be necessary. It was discovered that it was not possible to maintain the wetland elements in a flooded condition. The original program hydrologic algorithms placed no limit on the magnitude of the Active Groundwater storage compartment. With the hydrologic conditions simulated and the time step used in this study, this storage tended to increase steadily, draining the upper soil layers. This resulted in negligible surface layer storages and virtually eliminated overland flows, so important in the floodplain wetland systems being simulated. Even the upland subbasins did not function properly until an upper limit was placed on the total inflow to the lower and active groundwater layers in order to keep the storages within expected limits. The algorithm implemented to limit this downward flow, now passes all inflow to the lower soil horizons through the Upper Zone storage. The original model included this a bypass as a means of allowing a volume of flow greater than the available upper zone storage to be routed downward directly to the Active Groundwater compartment.

With this modification, land surfaces may saturate completely under flooded conditions and will still function properly under drier conditions. All downflow was routed directly through the upper zone storage compartment, eliminating the bypass to the Active Groundwater horizon. The model originally provided this bypass to satisfy Lower Zone demand that exceeded the size of the Upper Zone storage capacity, particularly when the Upper Zone is modeled as a relatively thin layer. In order to correct an algorithm deficiency related to the large surface layer storages simulated, subroutine UZINF was modified to compare and limit the calculated Upper Zone inflow to the maximum inflow allowed by the algorithm. This problem was caused by operation with larger surface layer storages than the original algorithm was developed to simulate, and is otherwise unrelated to the modification controlling the total percolation to the lower layers.

These modifications required changes in PWATER, DIVISN, and UZINF. The lower zone storage limitation is now implemented by specifying the size of the active groundwater storage as the third parameter in table PTABLE-PARMS. The maximum lower zone storage is now the sum of:

- Three times the Upper Zone nominal storage (UZSN),
- Three times the Lower Zone nominal storage (LZSN), and
- A new input parameter, the Active Groundwater storage.

The net effect of these modifications was that the upland-type systems function as they did previously with the original surface routing algorithm until the soil became completely saturated, in which case somewhat greater surface runoff would be stimulated. No problems were experienced in satisfying the limited lower zone water demands by the modified method. A similar result could have been accomplished by arbitrarily reducing the infiltration capacity of the soils. However, this approach would not have produced the same overall effect because the soils found in the study area are generally very porous, have a limited available storage capacity, and quickly become saturated when sufficient moisture is available.

These modifications were considered to be essential to produce reasonable hydrologic functioning of the wetland systems. The hydrologic algorithm modifications had no effect on the subsequent quality calculations, since the nutrient fluxes were already simulated in this manner in Section MSTLAY. Figure 2 illustrates graphically the program modifications and enhancements made to Section PWATER.

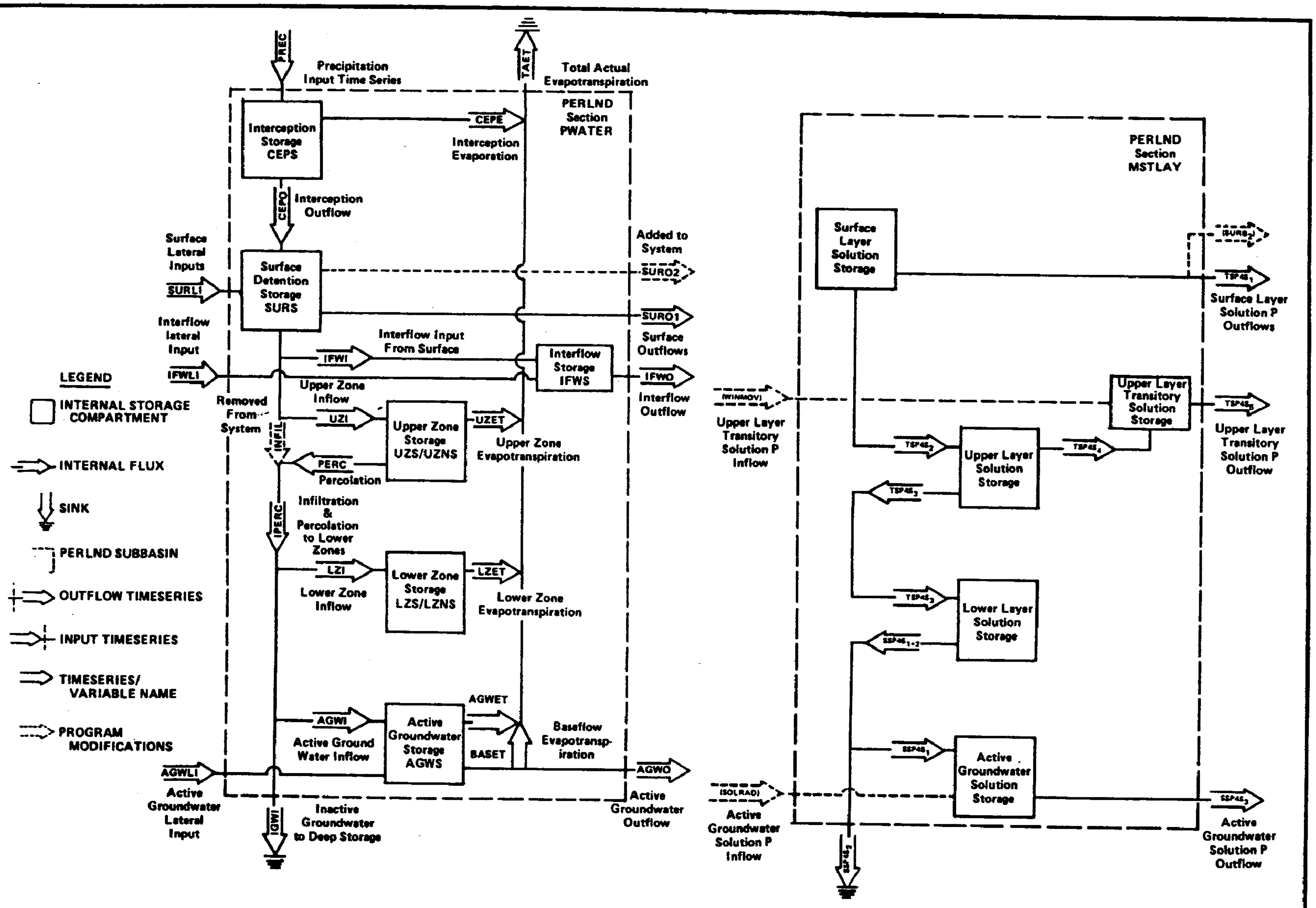
PREC
Precipitation Input Time Series
TAET
Total Actual Evapotranspiration
PERLND Section PWATER
Interception Storage CEPS
CEPE
Interception Evaporation
CEPO
Interception Outflow
Surface Lateral Inputs
SURLI
Surface Detention Storage SURS
Added to System
SURO2
SURO1
Surface Outflows
Interflow lateral Input
IFWLI
IFWI
Interflow Input From Surface
Interflow Storage IFWS
IFWO
Interflow Outflow
Upper Zone Inflow
UZI
Removed From System
INFIL
Upper Zone Storage UZS/UZNS
UZET
Upper Zone Evapotranspiration
PERC
Percolation
IPERC
Infiltration & Percolation to Lower Zones
LZI
Lower Zone Inflow
Lower Zone Storage LZS/LZNS
LZET
Lower Zone Evapotranspiration
AGWI
Active Ground Water Inflow
Active Groundwater Storage AGWS
AGWET
BASET
Baseflow Evapotranspiration
AGWLI
Active Groundwater Lateral Input
AGWO
Active Groundwater Outflow
IGWI
Inactive Groundwater to Deep Storage
LEGEND
INTERNAL STORAGE COMPARTMENT
INTERNAL FLUX
SINK
PERLND SUBBASIN
OUTFLOW TIMESERIES
INPUT TIMESERIES
TIMESERIES/ VARIABLE NAME
PROGRAM MODIFICATIONS
PERLND Section MSTLAY
Surface Layer Solution Storage
Surface Layer Solution P Outflows
WINMOV
Upper Layer Transitory Solution P Inflow
Upper Layer Transitory Solution Storage
Upper Layer Transitory Solution P Outflow
Upper Layer Solution Storage
Lower Layer Solution Storage
Active Groundwater Solution Storage
SOLRAD
Active Groundwater Solution P Inflow
Active Groundwater Solution P Outflow

FIGURE 2.

NUTRIENT SIMULATION ENHANCEMENTS

Initial review of the applicable program algorithms indicated several deficiencies in the agrichemical section PHOS of the PERLND module. Although a plant phosphorous uptake mechanism to remove phosphorous from the solution storage was available, no corresponding dieback mechanism existed. As a result, the plant phosphorous storage compartment (PLTP) functioned as an ever increasing dead-end storage. In the systems being simulated, plant phosphorus cycling was considered important enough to warrant the inclusion of a plant phosphorous dieback mechanism in order to recycle phosphorous to the organic detrital storage.

The original program was only capable of simulating the downward and outward flows of nutrients, the upward transport of nutrients between soil horizons was not possible. However, plants function as nutrient "pumps", transporting nutrients to the surface layer from the lower soil horizons via their root systems. This was simulated by allowing phosphorus uptake to occur within the various soil horizons, adjusting the plant phosphorous layer storages based on ratios of the aboveground to the soil horizon storages. The overall ratio of aboveground to belowground biomass may be varied monthly.

The original program plant uptake algorithm calculated plant phosphorous uptake as a first-order reaction on the basis of the available solution phosphorous storage, independent of the magnitude of the plant storage compartment. In the wetland systems simulated, the magnitude of the storage compartments of surface layer "soil" phosphate phosphorus in solution often fluctuate widely. This resulted an unstable simulation of the plant phosphorus storage compartment. To remedy this situation, the demand basis for the plant uptake from the solution compartment was modified to depend on the existing plant phosphorous storage.

Mineralization rates in the surface layer corresponding to oxidizing (aerobic) or reducing (anaerobic) conditions can now be selected for use by the program based on the degree of surface flooding.

Examination of the program capabilities revealed that nutrient loading input time series capabilities for the PERLND module agrichemical section was inadequate. The existing mechanism for external nutrient loadings, SPEC-ACTIONS, was found to be unsuitable because the data format used by SPEC-ACTIONS was foreign to all other program systems, the SPEC-ACTIONS data format cannot be generated by the program in time series format, SPEC-ACTIONS is not operational in the networking portion of the system, and performs a direct memory address variable modification, which may change with program modification. As a result, a set of phosphorous input time series were developed to facilitate the simulation. These time series also provide the nutrient flow routing capabilities lacking in the original program. The surface layer input now serves as the input point for all external nutrient loadings, such as from adjacent subbasin inflows, adjacent channel overbank inflows, atmospheric fallout, fertilizer inputs, etc. Program modifications affecting the nutrient simulation are illustrated in Figure 2 for Section

MSTLAY and solute transport and in Figure 3 for Section PHOS intra- and inter-compartmental fluxes.

The original networking scheme used to input solution phosphorous inflow time series corresponding to the various hydrologic flows placed these inflows directly into the solution phosphorous storage compartment. As a result of the sequence in which calculations are performed by HSPF, the phosphorous loadings were found to be quickly transported out of the system or incorporated into the organic phosphorous or plant phosphorous storages. The surface layer phosphorus inputs are now added directly to the phosphate adsorbed to the soil in the surface layer. This implies the soils reactions attain equilibrium immediately, avoiding the flushing and rapid uptake phenomena. The assumption is that phosphorus inputs rapidly become bound to the soil. The resulting model is also operationally more robust than the original method.

These difficulties were alleviated by the redirection of three existing input time series from the unused SNOW section to the PHOS section of the PERLND module. The DTMPG, SOLRAD, and WINMOW time series now serve as surface, interflow and active groundwater layer solution phosphorous inflow times series, respectively. These correspond directly to the hydrologic flow time series in these layers and provide greatly expanded networking capabilities. An additional nutrient outflow time series (IFWOB) was converted from its original function and added to the PHOS section to correspond to the second outflow added to the surface flow routing capability of the PWATER section.

The modifications described above required program changes in subroutines PHOS, PHORXN, MSTLAY, TOPLAY, TOPMOV, PWATPB, and PWAACC and effectively disabled the SNOW section of the PERLND module. Modifications were also required in the external file INFOFL. No additional input data was required to use the modified time series DTMPG, WINMOW, SOLRAD, and IFWOB. They were handled exactly like the existing time series in the TSS and in the EXT SOURCES, NETWORK and EXT TARGETS blocks to perform their new functions.

SUMMARY

The HSPF program is somewhat unsophisticated hydrologically, having been developed around a number of empirically derived algorithms. The program was considerably modified in an attempt to reduce any potential impacts which might have resulted from some of the hydrologic algorithm deficiencies. More serious structural limitations of HSPF were first, the model is one-dimensional and second, flows are assumed to be unidirectional. The one-dimensionality of the model necessitated simulating floodplains as flat, amorphous regions with constant surface areas. Thus the program can only determine whether a particular part of the system is inundated and to what depth.

Flow through the model is constrained to be unidirectional, since momentum is not taken into consideration. It is assumed that an element cannot hydrologically or hydraulically influence an element upstream from it and

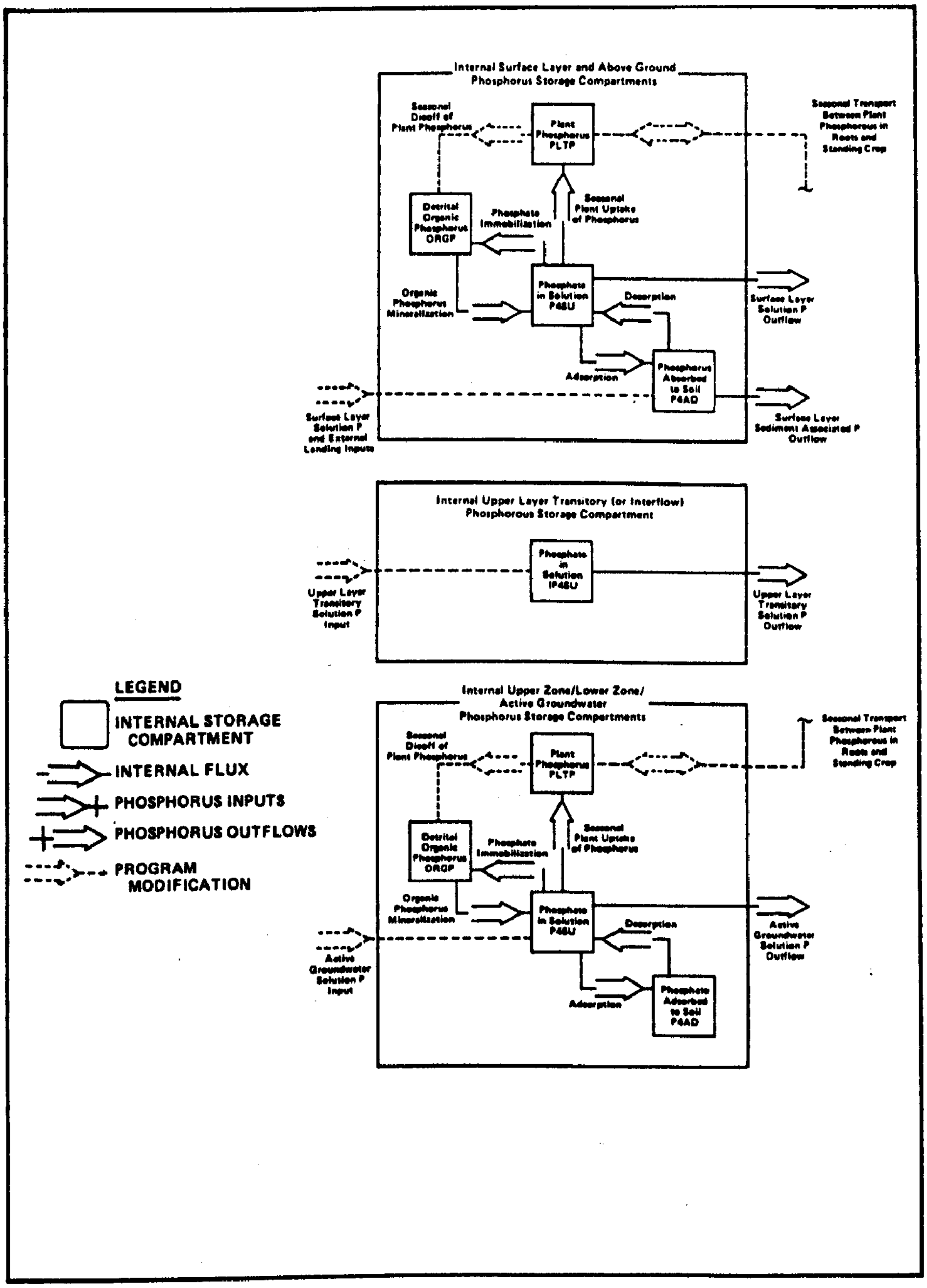
Internal Surface Layer and Above Ground Phosphorus Storage Compartments
Seasonal Dieoff of Plant Phosphorus
Plant Phosphorus PLTP
Seasonal Transport Between Plant Phosphorous in Roots and Standing Crop
Detrital Organic Phosphorus ORGP
Phosphate Immobilization
Seasonal Plant Uptake of Phosphorus
Organic Phosphorus Mineralization
Phosphate in Solution P4SU
Desorption
Surface Layer Solution P Outflow
Adsorption
Phosphorus Absorbed to Soil P4AD
Surface Layer Solution P and External Landing Inputs
Surface Layer Sediment Associated P Outflow
Internal Upper Layer Transitory (or Interflow) Phosphorous Storage Compartment
Phosphate in Solution IP4SU
Upper Layer Transitory Solution P Input
Upper Layer Transitory Solution P Outflow
LEGEND
INTERNAL STORAGE COMPARTMENT
INTERNAL FLUX
PHOSPHORUS INPUTS
PHOSPHORUS OUTFLOWS
PROGRAM MODIFICATION
Internal Upper Zone/Lower Zone/ Active Groundwater Phosphorus Storage Compartments
Seasonal Dieoff of Plant Phosphorus
Plant Phosphorus PLTP
Seasonal Transport Between Plant Phosphorous in Roots and Standing Crop
Detrital Organic Phosphorus ORGP
Phosphate Immobilization
Seasonal Plant Uptake of Phosphorus
Organic Phosphorus Mineralization
Phosphate in Solution P4SU
Desorption
Active Groundwater Solution P Outflow
Active Groundwater Solution P Input
Adsorption
Phosphate Adsorbed to Soil P4AD

FIGURE 3.

that downstream conditions, external to the model, cannot affect the hydrologic or hydraulic behavior of the system. In this study, outflows from reach and wetland subbasin elements were assumed to be functions of volume or depth. The methodology developed to define these relationships utilized the HEC-2 program, and an analysis of historical streamflow records. Using this methodology, it was possible to approximate the complex routing of surface water flows and attendant phosphorus loads between floodplain wetlands and the restored or channelized Kissimmee River over a wide range of hydrologic conditions. Thus many of the limitations of the hydrologic and hydraulic structural simplifications and assumptions of HSPF were circumvented.

The resulting model framework was such that those alternatives which route more water with its attendant phosphorus load to the "wetland" subbasins, predict lower downstream phosphorus loadings. In effect, the simulation models predict that downstream phosphorus loads are to be closely related to the hydrology of the impacted wetland areas. This supports the popular notion of the value of wetlands in regards to their nutrient removal capability. Thus, if the hydrologic/hydraulic assumptions which have been incorporated into the framework of the simulation models result in an improper allocation of either the absolute amount of water outside the channel or the time that such water spends out of the channel, it can be anticipated that a corresponding reduction in the capability of the model to accurately simulate phosphorus dynamics within the system will result.

Another major constraint was that the simulation of the system was limited to one year. While this appears to be adequate for comparison of alternatives, the simulations say nothing about the long-term viability of any of the alternatives regarding phosphorus removal and retention. For any alternative to continue to effectively remove phosphorus in the long run, there must be long-term "sinks" or storage compartments for phosphorus within the system. However, the simulation models for the various restoration alternatives, were not specifically designed to address this issue and no conclusions can be drawn from the results regarding the long-term phosphorus removal effectiveness of any of the alternatives.

SIMULATION OF A REGIONAL WATER SUPPLY WITH AQUIFER STORAGE

by: Ronald L. Wycoff, P.E.
CH2M HILL
Gainesville, Florida 32601

ABSTRACT

The storage of treated water in a suitable aquifer for later recovery and use may be an economically feasible component of a regional water supply system. This concept, known as aquifer storage recovery (ASR), can be used to reduce the required treatment and/or raw water storage capacities of a water supply system such that a net production cost savings is achieved. This paper describes a computer simulation model constructed for the General Development Utilities (GDU) Peace River Regional Water Supply System, located near Fort Ogden, Florida. Principal hydrologic components of the simulation include streamflow records extension and statistical analysis, generation of synthetic streamflow traces, and simulation of raw water diversions subject to regulatory, water quality, and pumping constraints. Major components of the water supply system considered include the water treatment plant, off-line surface storage of raw river water, and underground storage of treated water. The purpose of the simulation is to develop relationships between the size of major system components and overall system reliability. These relationships, along with component cost data, will be used in future analysis to identify the least-cost combination of major system components that will meet design demands at an acceptable reliability.

INTRODUCTION

The GDU Peace River Water Supply Facility is located on the west bank of the Peace River, near Fort Ogden, Florida. Total tributary area at the raw water intake is approximately 1,787 square miles, and average daily streamflow is approximately 1,540 cfs (990 mgd). All of the tributary area except for about 70 square miles is gaged by three U.S. Geological Survey (USGS) stream gages. The recording stations are the Peace River at Arcadia, Joshua Creek at Nocatee, and Horse Creek near Arcadia. Records are available beginning in 1932 for the Peace River gage and in 1951 for Joshua

Creek and Horse Creek. The locations of the three USGS stream gages, as well as the water treatment plant diversion point, are shown in Figure 1.

Existing water supply facilities include a diversion pumping station with a maximum capacity of 22 mgd, a 6-mgd water treatment plant, and an off-line raw water storage reservoir with an effective storage volume of 1,920 acre-feet (625 mg). These facilities serve a rapidly developing area of southwest Florida. Ultimate maximum day water demand for the Peace River facility is projected to reach 60 mgd by the year 2011 (1).

The planning and design of water supply facilities to meet these future demands in an economically optimum manner represents a significant challenge. The objective of the current project is to investigate the hydrologic and economic feasibility of ASR at the Peace River plant site. The project consists of two major parts. The first part involves construction and operation of test wells to quantify the storage characteristics and hydraulic response of the potential storage zones, as well as the background quality of the native groundwater. The second part of the project consists of development and application of the water supply systems simulation model presented in this paper. This work is ongoing, and all simulation techniques and results reported herein should be considered preliminary and subject to change.

PEACE RIVER WATER SUPPLY SIMULATION MODEL

The Peace River water supply simulation model includes two major modules, which operate independently and sequentially to ensure flexibility in the analysis. The main objective of the first module, known as PEACE, is the calculation of potential diverted streamflows (quantity and quality) from the historical and/or generated total flow time series. The second module, known as PLANT, utilizes these flow and quality arrays as inputs for the simulation of the Peace River Water Supply System, including the evaluation of the reliability of the overall system.

For this project, total dissolved solids (TDS) was chosen as the water quality parameter of primary interest. Preliminary analysis of the raw Peace River water as well as the native groundwater indicates that the drinking water standard for TDS (i.e., 500 mg/L, maximum) would likely be exceeded before any other drinking water limits were violated. Therefore, TDS is considered to be the controlling water quality parameter, and the term "quality", in this paper, refers to TDS concentration.

THE PEACE MODULE

The basic computational sequence for the PEACE module is summarized in Figure 2. The main input data are the historical monthly flows and the regression equation parameters for flow record extension and river quality (TDS) calculation. Monthly flows at the diversion structure are calculated first; then, optionally, the corresponding potential diverted flows and their quality are calculated using subroutines DIVERS and QUALTY, respectively. Both of these time series are then saved on an off-line computer

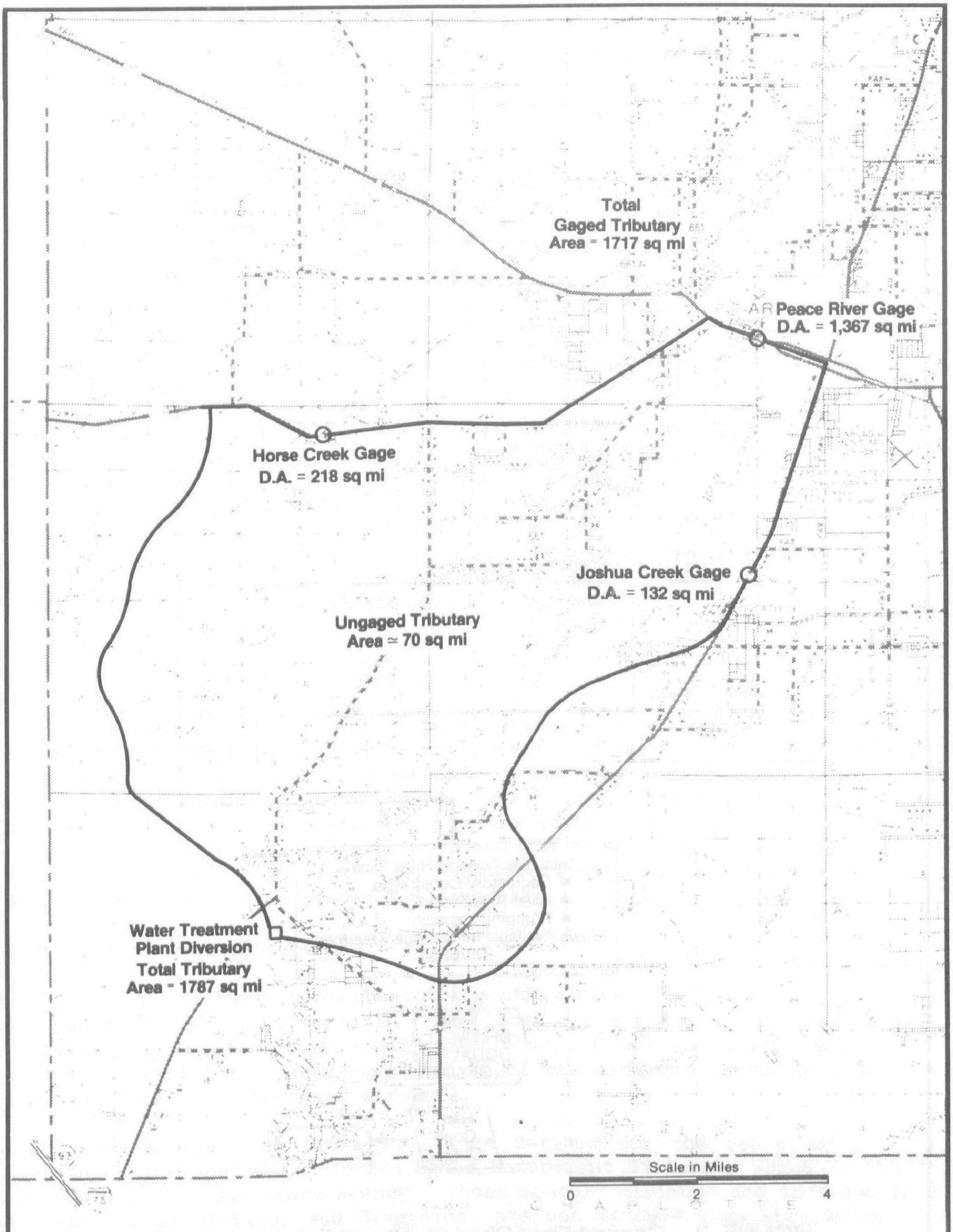

FIGURE 1. Location of Streamflow Recording Stations and Water Treatment Plant Diversion.

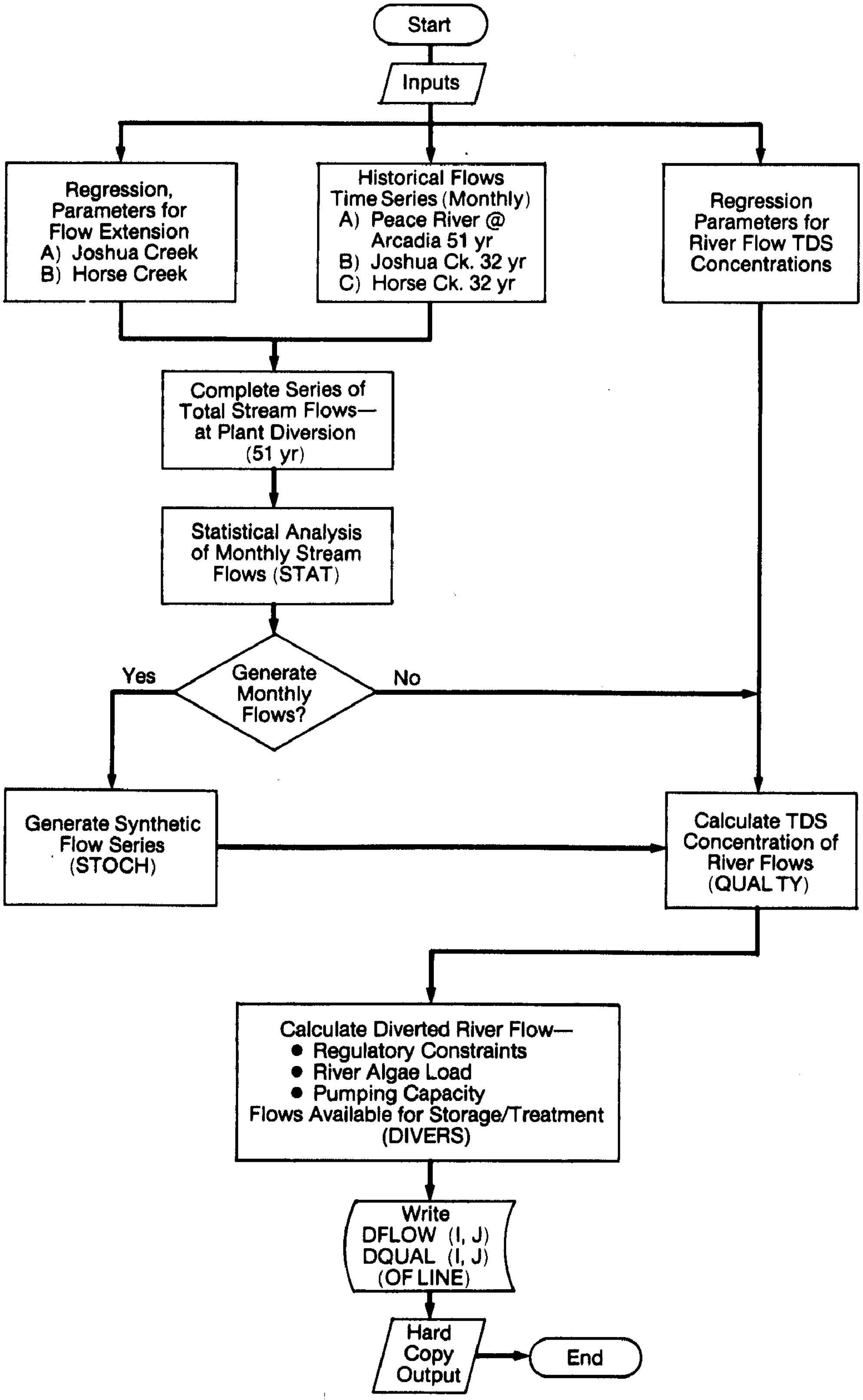

FIGURE 2. General Flow Chart for Streamflow Portion of Water Supply Simulation (PEACE—Module).

file for later use as input to the PLANT module. If generated flows are needed for further simulation, the subroutine STAT is used for statistical analysis of the historical total flows and the estimation of the parameters for the flow generation module (STOCH). The optional stochastic generation of monthly streamflows is based on a first-order Markov Model. TDS and diverted monthly flows are then calculated by subroutines QUALTY and DIVERS, respectively. All time series defined within the PEACE module may be printed, along with their statistical summaries, by calling the STAT subroutine.

Observed Flow Records Extension and Adjustment

Two simple power functions are used for the flow record extension at the Joshua Creek and Horse Creek stations, where observed monthly flows from 1932 to 1950 are unavailable. Regression equation parameters were estimated using the 384 monthly observations of concurrent streamflow data extending from 1951 to 1982. These parameters are estimated by simple linear regression on the logarithms of the observed concurrent monthly flows. The resulting equations are then used to estimate monthly streamflow at Joshua Creek and Horse Creek, based on the observed monthly flow at the Peace River Arcadia gage for water years 1932 through 1950. Monthly flows at all three stations are then summed, and this total is increased 4.1 percent to account for the ungaged tributary area (see Figure 1). The resulting flow series is considered an observed 51-year streamflow trace at the Peace River treatment plant diversion structure.

Computation of River TDS Concentration

The monthly river water TDS concentrations are related to the total monthly river flows by a quality rating curve which is also represented by a simple power function. The parameters of the TDS rating curve were estimated using 4 years of flow and TDS data from the Peace River at Arcadia gaging station. These parameters are estimated by simple linear regression of the logarithms of the concentrations and the flows. The resulting rating curve is applied to the observed or generated monthly streamflow series in order to establish the estimated TDS concentration of each monthly flow.

Computation of Available Flows

Only a small fraction of the total monthly river flow is available for water supply purposes. Divertable river flows are a function of three major constraints, as well as the total river flow. These are a regulatory low-flow constraint, the pumping capacity of the diversion structure, and the algae content of the river water.

Minimum monthly low flows have been defined for the Peace River diversion by the Southwest Florida Water Management District, which is the responsible state regulatory agency. These monthly minimums are included in the diversion simulation, and diversions are not allowed when streamflows are less than these values.

Maximum diversion rates are controlled by the pumping capacity of the diversion structure, which is currently 22 mgd. However, other maximum rates may be simulated. In addition, the Peace River is subject to periodic algae blooms in the spring and summer months. During these algae blooms, river water is not diverted in order to prevent operational problems in the raw water storage reservoir and plant. This situation is simulated with the aid of a simple monthly probability distribution. The monthly probabilities of not diverting river water due to high algae content were determined by analysis of plant operating records and concurrent river algae data.

Synthetic Streamflow Generation

For water supply system analysis, more than one trace of flow series may be required for the assessment of reliability of the different components of the system under investigation. Given the limited length of historical records, the generation of synthetic traces of monthly flows series "similar" to the historical one has been included in the PEACE module as a user option. For this study, a first-order Markov Model was chosen (2, 3). The parameters of the stochastic model are computed by the PEACE module using the extended and adjusted flow record at the diversion point discussed previously.

THE PLANT MODULE

The PLANT module is essentially a flow and quality mass balance accounting procedure operating on a monthly time step. Computations are governed by a predefined flow distribution logic, which is given in Figure 3. Also shown in Figure 3 is a schematic diagram of the proposed Peace River Water Supply System. This system consists of five major components: 1) the river diversion structure; 2) the water treatment plant; 3) the aquifer storage/recovery system, which stores treated water in the aquifer; 4) the surface storage reservoir, which stores raw water in an off-line open storage basin; and 5) product water distribution.

The diversion of flows from the Peace River is addressed in the PEACE module, and these flows, therefore, are a known input to the PLANT module. They are referred to as "potential" diverted flows because they may or may not actually be diverted, depending upon the status of the surface storage reservoir at a given time step.

Flow distribution in any time step is a function of the potential diverted river flow available and the system demand. If the available diverted river flow is greater than the demand, then the river flow is treated and distributed, and the remaining available flow is treated and stored underground or stored as raw water in the surface reservoir, depending on the status of system components.

If the available diverted flow is less than the demand, then the deficit is obtained from the aquifer storage/recovery system if the water quality goal can be met. If the water quality goal cannot be met, then a blend of aquifer storage/recovery water and surface storage water is used that will meet the water quality goal. If this is not possible (i.e.,

FLOWS DISTRIBUTION LOGIC

A) DFLOW (I,J) ≥ DEMAND (J)

1) Treat & Distribute - [DEMAND (J)]

2) Treat & Inject - [QPLANT-DEMAND(J)]

3) Remaining Flow [if any] to SSR.
If SSR is full then water is lost
from system.

B) DFLOW (I,J) < DEMAND (J)

1) Treat DFLOW (I,J) & distribute

2) Treat additional flow from SSR to
maintain minimum plant flow if necessary.
QSSR = PLANTCM - DFLOW (I,J)

3) Obtain remaining required flow from ASRS.

4) Check quality of product water.
If quality criteria is not met, then
solve for blend of SSR waters and ASRS
waters which will meet quality goal.

5) If quality goal cannot be met then
satisfy DEMAND (J) from ASRS.
(This is a system failure.)

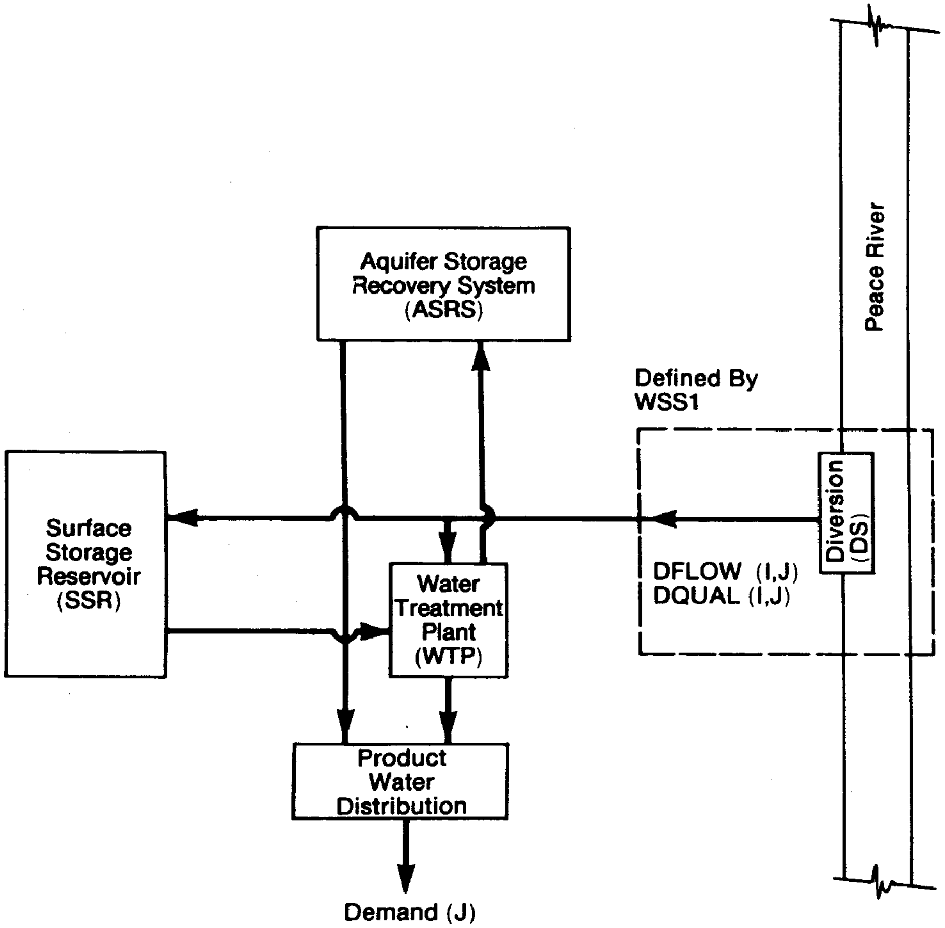

FIGURE 3. Schematic Diagram of Peace River Storage Treatment System.

surface reservoir empty), then the required water is obtained from the aquifer storage/recovery component, even if the water quality goal cannot be met. In this case, a failure occurs when the water supply system does not produce the desired quality water. Since native groundwater is available for pumping during extreme droughts, the system is fail-safe hydraulically, assuming that installed well field capacity matches demand.

This flow distribution logic is based on two basic assumptions. First, the aquifer storage/recovery system will be the primary means of water storage. The raw water surface reservoir will serve as a secondary or backup storage system. Second, the system will be operated in a manner that maximizes its reliability from a water quality standpoint. That is, if it is physically possible to deliver water within the quality goal, such water will be produced and delivered.

Aquifer Storage/Recovery System Mixing

A unique element of this simulation is the aquifer storage/recovery system. When treated water is injected into the aquifer, it will displace and mix with the native groundwater. Therefore, when waters are withdrawn from the aquifer, they will be a blend of the treated injected waters and the native groundwaters. The quality of the recovered water, therefore, will be a function of this mixing process. The approach utilized in PLANT consists of development of an empirical or conceptual model rather than a theoretical model. Data used to develop this mixing model were taken from onsite aquifer storage/recovery testing.

The components of the aquifer storage/recovery system mixing model are shown schematically in Figure 4. Three major components are identified: native water storage, injected water storage, and a mixing process. Inputs to the aquifer storage/recovery subsystem include the monthly time series of injection volumes (VI(I,J)) and corresponding quality (QI(I,J)). Outputs include the monthly time series of recovered volumes (VR(I,J)) and corresponding quality (QR(I,J)).

Native water is assumed to occupy an infinite reservoir and have a constant quality (QN). Therefore, the volume of native water will never be depleted, nor will its quality vary with time.

When treated water is injected into the aquifer, it will displace the native waters near the injection wells. The maximum volume available for storage of injected waters is unlimited. However, the amount in storage in any given time step (RRV(I,J)) will be the summation of all waters previously injected, less the summation of all injected waters previously recovered. The quality of the injected waters (RRQ(I,J)) is assumed to be the composite quality of all injected waters in storage at any given time.

The blend of injected and native waters withdrawn from the aquifer during a recovery period is computed by application of an exponential relationship as illustrated in Figure 4. This empirical relationship does not simulate the mixing process itself, but only defines the results (i.e.,

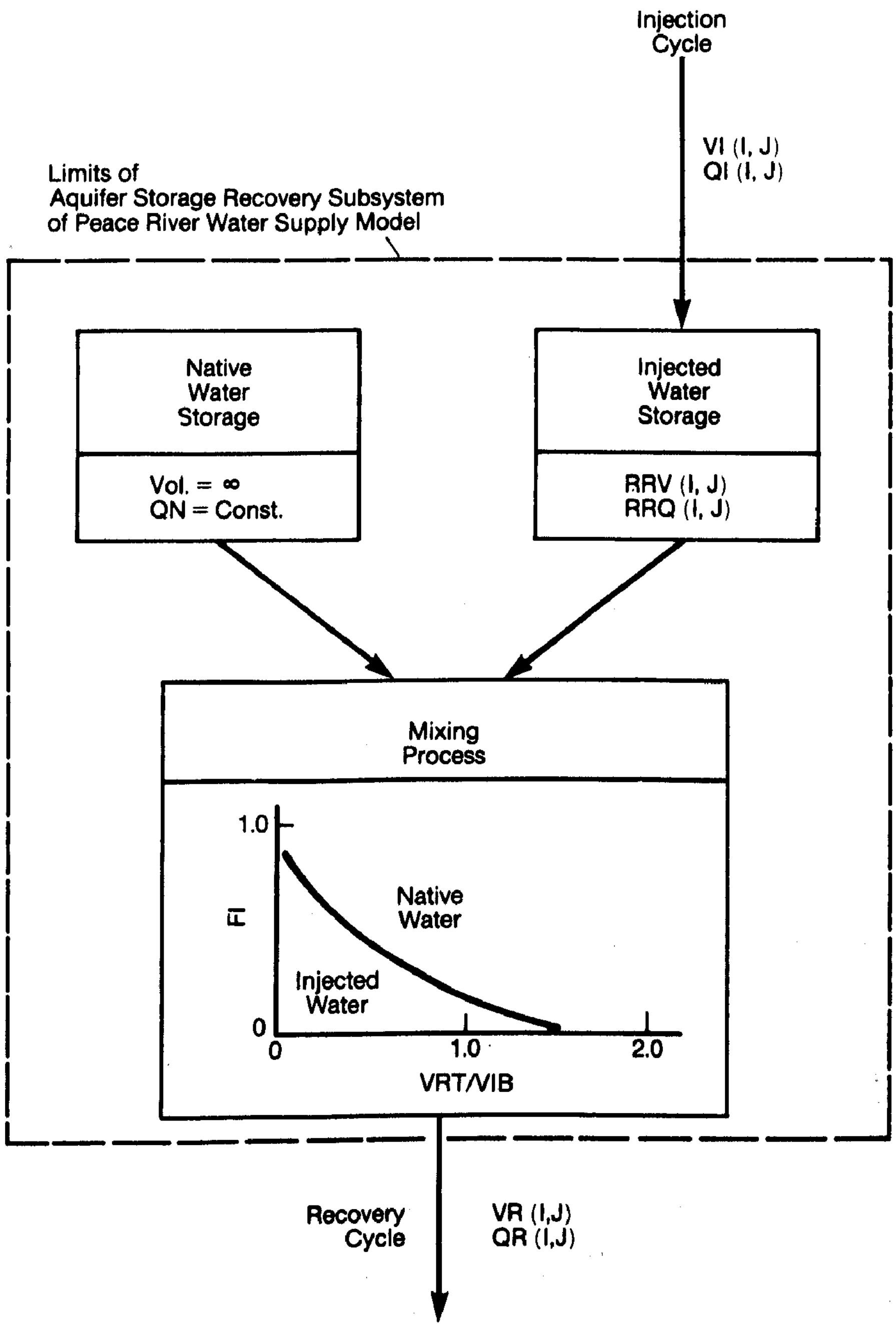

FIGURE 4. Conceptual Sketch of Aquifer Storage Recovery Mixing Model.

response) of the aquifer mixing. The relationship and its parameters are defined as follows:

$$FI = \alpha\, e^{\beta(VRT/VIB)} \qquad (1)$$

Where:

FI = instantaneous fraction of injected water contained in recovered water mixture

VRT = total volume recovered since beginning of recovery cycle

VIB = volume of injected water in aquifer storage at beginning of recovery cycle

α and β = aquifer mixing parameters defined from analysis of observed onsite aquifer storage/recovery test data

<u>Raw Water Storage Reservoir</u>

The surface reservoir simulation is a simple mass balance of both the quantity and quality of raw river water. Inflows accounted for include diverted river flow and direct rainfall. Outflows include water supply withdrawals and evaporation. Both rainfall and evaporation are defined on a monthly basis. If the reservoir is full and potential diverted flows are available, then these flows are lost from the system and will not be available at a later time step.

RESULTS

To date, the simulation has been applied to define relationships between treatment plant/ASR capacity, off-line raw water storage capacity, and overall system reliability for the year 2011 conditions (i.e., 60 mgd maximum day and 37.5 mgd average daily flow). Twenty different combinations of treatment capacity and off-line storage capacity have been simulated and a family of system reliability curves developed.

Preliminary economic analysis, based on the results of the simulation as well as an estimated annual cost of alternative systems, indicates substantial overall costs savings using ASR when compared to the traditional surface-water supply approach, which utilizes raw water storage only. Savings on the order of 25 percent using ASR appear feasible. These savings result from a reduction in required treatment capacity as well as in raw water storage volume. However, at this time, the overall best solution using ASR is not obvious from the economic analysis because differences between alternatives are small. The final selection will involve the evaluation of many factors in addition to system economics and reliability.

Future plans for this work include various refinements in the simulation, as well as application to design demand levels between the current system capacity and the ultimate (i.e., year 2011) system demand.

ACKNOWLEDGMENT

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SIMULATION OF POSSIBLE EFFECTS OF DEEP PUMPING ON SURFACE HYDROLOGY USING HSPF

by: C. Nancy Hicks, Wayne C. Huber
and James P. Heaney
Department of Environmental Engineering Sciences
University of Florida
Gainesville, Florida 32611

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University of Florida
Gainesville, Florida
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ABSTRACT

The continuous simulation program HSPF is being used to model the hydrology of the Cypress Creek watershed in Pasco County, Florida north of Tampa. Various strategies have been considered for simulating possible effects on surface water of pumping 30 mgd from deep wells within the Floridan Aquifer beneath the watershed boundaries. Modeling complications include the fact that HSPF was not specifically designed to simulate withdrawals from groundwater by pumping; hence, indirect methods have to be applied. Other factors that affect the surface hydrology are also being considered, especially surface drainage activities and recurrent droughts.

CYPRESS CREEK WATERSHED

INTRODUCTION

Cypress Creek Watershed, located north of Tampa in Pasco County (Figure 1), has undergone extensive development over the last decade. Landowners in the basin have made major modifications of surface drainage, and the West Coast Regional Water Supply Authority has established a 30 mgd capacity wellfield in the center of the watershed, pumping from the deep groundwater aquifer. The Department of Environmental Engineering Sciences at the University of Florida has undertaken a two-year project to study the possible effects on the basin of these types of development under the varying meteorological conditions of the region. Recurrent droughts are a particular problem.

Previous studies of the basin include seven years of hydrobiological monitoring of the Cypress Creek Wellfield by the Southwest Florida Water Management District, Biological Research Associates and Conservation Consultants, Inc. (Rochow, 1983). Two models for simulation of steady state groundwater flow for a 932 square mile area containing Cypress Creek Wellfield and nine other municipal wellfields were developed by the USGS: a two dimensional model (Hutchinson et al., 1981) and a quasi-three-dimensional model (Hutchinson, 1984). In addition, a model patterned after the Prickett-Lonnquist aquifer simulation model is currently maintained by SWFWMD. Previous studies have been inconclusive as to the effects of either drainage development or deep aquifer pumping on the surface hydrology of the basin.

Cypress Creek Watershed consists of 117 square miles of sandy ridges, flatwoods, hammocks and swamps drained by Cypress Creek. The creek runs through an area of low-lying swamps and wetlands known as Big Cypress Swamp. The five soil associations generally found in the area contain soils which tend to be sandy throughout and are characterized by wetness, poor filtration, and ponding. These soils constitute the surficial zone and range from 20 to 40 feet in thickness (SWFWMD, 1982). Under these unconsolidated deposits is a clay layer between 2 to 25 feet in thickness which acts as a semipermeable confining layer (Ryder, 1978). Under the clay lies the consolidated rock of the Floridan aquifer.

BASIN RECORDS

Two rain gages located in the basin provide daily records for a period of seven years, 1977 to 1983. One gage located just east of the basin at St. Leo provides an hourly rainfall record from 1944 to the present. This gage gives a long term average annual rainfall for the area of 54 inches. Outside the western boundary of the watershed at Lake Padgett is a pan evaporation station with eleven incomplete years of record, 1972 to 1983; the mean annual pan evaporation for the seven complete years is 56 inches.

There are three stream gages on Cypress Creek which offer some discharge and stage data. The Worthington Gardens gage is at the mouth of the watershed with a period of record from June 1974 to the present. The Drexel gage lies just south of the Cypress Creek Wellfield and has been discontinued but provides records from 1977 to 1981. The San Antonio gage just north of the wellfield provides the longest record for the stream, from 1963 to the present. The various gaging stations are shown in Figure 1.

SIMULATION

OBJECTIVES

As stated, the objectives of the study are to look at the effects on the surface hydrology of both on-going drainage development and deep aquifer pumping. A long term continuous simulation model was needed to span the period prior to wellfield operation (pre-1976) to the present. A groundwater model could best simulate deep aquifer pumpage, but could not as accurately capture the effects of continuous changes in

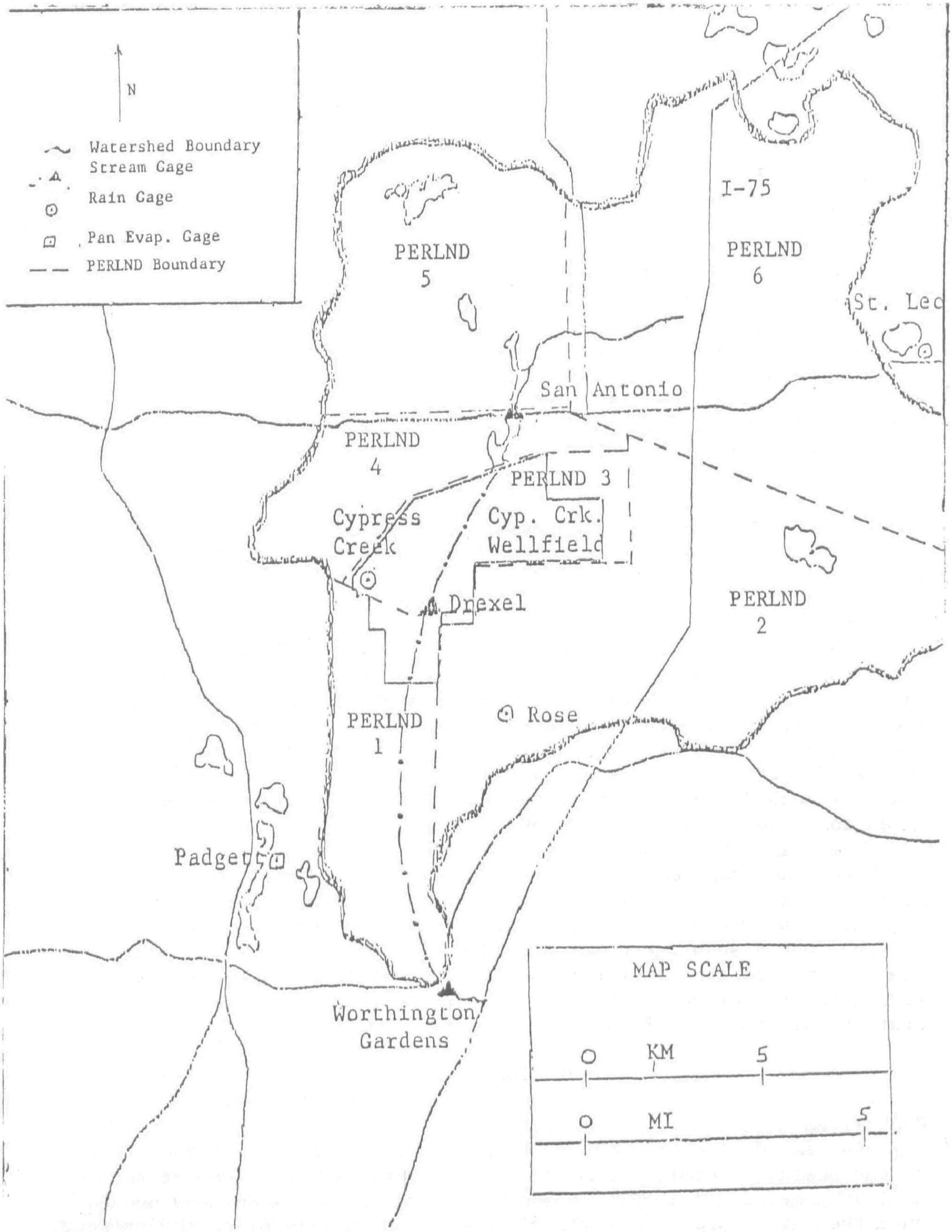


Figure 1. Gaging stations in the vicinity of Cypress Creek watershed.

surface drainage; the fact that the Prickett groundwater model of the basin was already in operation at SWFWMD was an added consideration. For these reasons, it was decided that a surface model could best allow for the examination and comparison of hydrographs and stages before and after various hydrologic modifications and conditions, which might include surface drainage, deep aquifer pumping, general development in the watershed, and droughts.

CHOICE OF MODEL

Given the aforementioned characteristics desired in the model, HSPF (Hydrological Simulation Program - Fortran) was chosen. The initial release of HSPF was prepared by Hydrocomp Incorporated. The revised version used for this project, Release 7.0, was prepared by Anderson-Nichols and Company and obtained through EPA's Environmental Research Laboratory in Athens, Georgia (Johanson et al., 1981). HSPF is a long-term continuous simulation model which is both well documented and well supported. It was expected that HSPF could simulate the surface hydrology well, although that has proved challenging for the swamp conditions existing in Cypress Creek Watershed.

CALIBRATION

Parameters

The HSPF component which controls the hydrologic simulation of the basin is called PWATER. In section PWATER there are over 25 parameters which are used in the description of each subcatchment. Cypress Creek Watershed was divided into 6 permeable land segments or sub-catchments, designated as PERLND 1 through PERLND 6 (Figure 1). A great difficulty arises in connecting over 150 parameters with numbers that are physically meaningful. Suggestions for parameter values according to watershed location and land use can be found in the user's manual for the ARM (Agricultural Runoff Management) Model. (Donigian et al., 1978). Some of the more important parameters affecting runoff volumes are the water storage capacities for the unsaturated soil zones of the surficial aquifer.

Storages

Water in the unsaturated zone can be divided into six storage blocks in HSPF (Figure 2). Interception storage (CEPS) is water retained on plant surfaces. Surface storage (SURS) is depression storage on the surface. Interflow storage (IFWS) contains water that flows below the surface to streams and other surface water bodies. The upper zone storage (UZS) is the upper few inches of the unsaturated zone from which water infiltrates to lower zone storage (LZS) or leaves as evapotranspiration. From the lower zone water may also leave by evapotranspiration or percolate into active groundwater storage (AGWS). Active groundwater may become runoff, evapotranspiration, or be lost to the simulation as deep percolation.

Each storage has a state variable which is given a value to reflect the amount of water in the soil at the beginning of the simulation period. The upper and lower zones, however, also have nominal storage capacity parameters whose values indicate the maximum amount of water which can be stored in these zones. These nominal capacities greatly influence the amount of runoff generated by the simulation and thus the entire water balance.

To determine the nominal storage capacities, the soil groups occurring in the basin were determined from the Soil Survey of Pasco County (Stankey, 1982), and the percent of each soil group occuring in each of the six land segments was estimated. SCS Soil Interpretation Sheets provided soil storage capacities for soils in each soil group. The SCS data divided the capacities into two horizons; the upper horizon consisted of the first 5 inches of soil and the lower horizon consisted of the soil from the 5 inch depth to the water table. It was decided that these two horizons would correspond well to the HSPF upper and lower zone storage blocks (Figure 2).

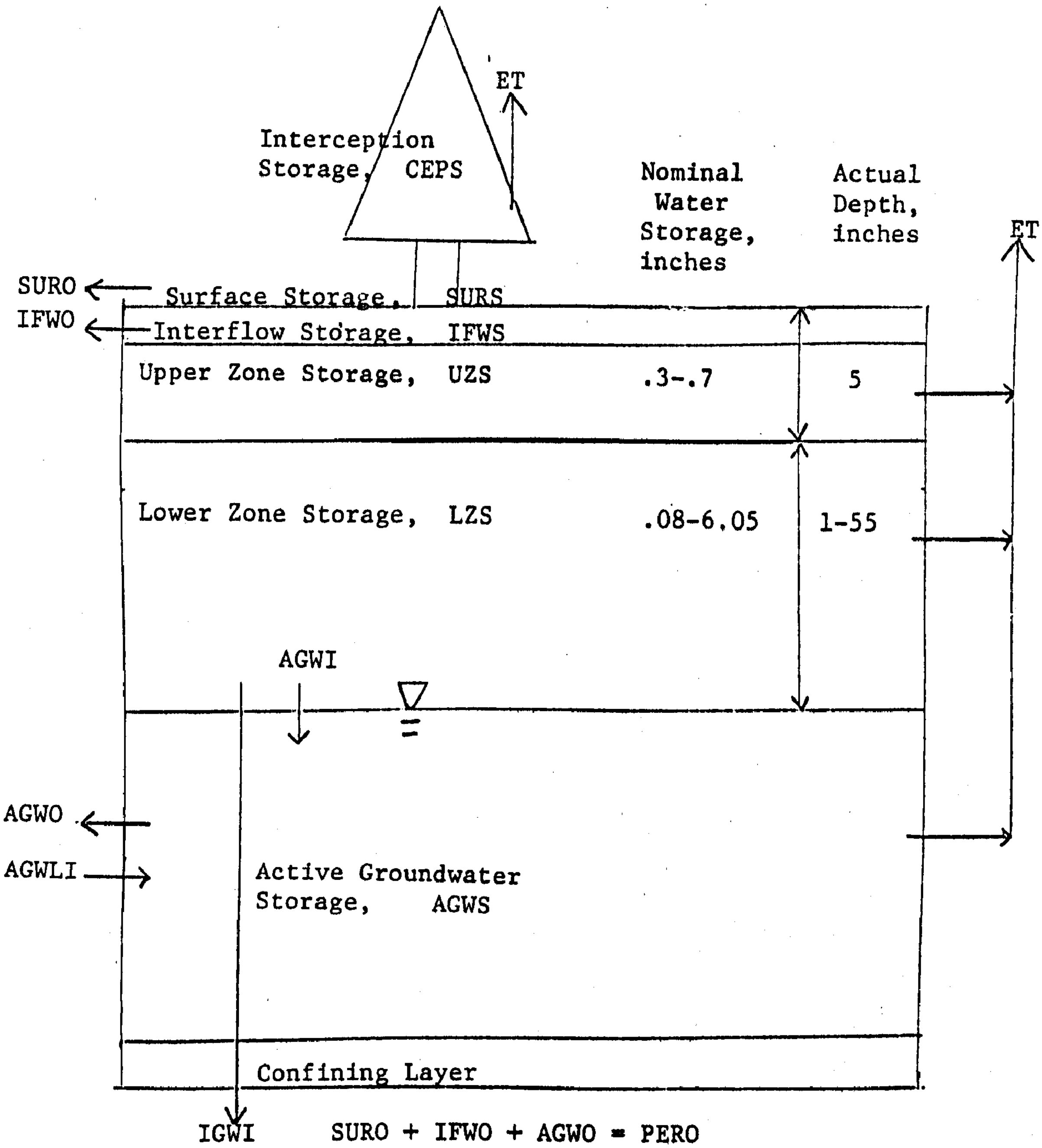
The horizon storage capacity information for each soil group was combined. Each group was then weighted as a percentage of its occurrence in each land segment and the storage capacities for each horizon were multiplied by the weights to determine total storage capacity horizons for each land segment. The final HSPF nominal storage capacities were obtained by multiplying by the soil zone depths, 5 inches for the upper zone horizon and the remaining inches between the 5 inch depth and the average water table depth for the lower zone horizon. Average depth to the water table was obtained for each soil group from the SCS soils data. The upper zone storage capacities ranged from 0.25 to 0.70 inches. Lower zone storage capacities ranged from 0.08 to 6.05 inches. For each land segment the minimum capacity for the upper or lower horizon was used because HSPF allows overfill of soil storages; the nominal capacities input into the model are indicators, not maximum capacities.

COMPARISON

Parameters reflecting the best available data about the watershed are shown for two land segments in Table 1. Some method of comparison of simulated values against measured or predicted values was then needed. Starting at San Antonio, Cypress Creek gage discharges data have been used to calibrate measured annual and mean monthly discharge volumes in acre-ft against simulated annual and monthly volumes.

The gage data were available in the form of daily flows in cfs. To obtain the mean monthly volume of runoff in acre-ft, the mean monthly discharge rate was multiplied by days per month and by the factor 1.983 acre-ft per cfs-day. The annual volume was the total of the 12 mean monthly volumes over the course of a water year, October 1 through September 30.

Four methods were used to compare the measured and simulated volumes. Table 2 shows the simulated and measured values indicating which water years were being over- or under-predicted and to what extent. Then, three plots were generated using SAS (Statistical Analysis System) on the University of Florida computers. The first (Figure 3) was a scattergram of monthly measured volumes versus monthly simulated volumes; best fit would occur if the scatter of points approximated a 45 degree line. Next, a hydrograph was prepared of the monthly volumes versus time, with measured and simulated values overlaid (Figure 4); here, the emphasis would be to match timing and volume of flow, and peaks if possible. The third plot (Figure 5) was a double mass curve of measured versus simulated values, again looking for a 45 degree line. The results of the closest simulation to date for the upper third of the basin are shown in Figures 3, 4 and 5.



SURO + IFWO + AGWO = PERO

PERO = total runoff

IGWI = DEEPFR · (IGWI + AGWI)

Figure 2. Storage blocks for permeable land segments in HSPF

Table 1. PWATER PARAMETERS DESCRIBING LAND SEGMENTS 5 AND 6

Parameter	Units	Value 5	Value 6	Description
Interception:				
CEPSC	inches	0.15	0.15	Interception Storage Capacity
CEPS	inches	0.0	0.0	Initial Interception Storage
Surface Detention:				
SURS	inches	0.0	0.0	Initial Surface Detention Storage
Interflow:				
IFWS	inches	0.0	0.0	Initial Interflow Storage
Upper Zone:				
UZSN	inches	0.25	0.30	Upper Zone Nominal Storage
UZS	inches	0.017	0.001	Initial Upper Zone Storage
Lower Zone:				
LZSN	inches	3.30	4.40	Lower Zone Nominal Storage
LZS	inches	2.99	4.43	Initial Lower Zone Storage
Active Groundwater:				
AGWS	inches	1.51	2.04	Initial Active Groundwater Storage
Infiltration and Perolation:				
INFILT	in/hr	6.0	10.0	Index to Mean Infiltration Rate
INFILD	-	2.0	2.0	Ratio Max/Min Infiltration Rate
IRC	/day	0.9	0.9	Interflow Recession Rate
AGWRC	/day	0.985	0.985	Active Groundwater Recession Rate
DEEPFR	-	0.25	0.30	Fraction of Groundwater to Deep Aquifer
Evapotranspiration:				
FOREST	-	0.3	0.3	Fraction Winter Forest Transpiration
BASETP	-	0.0	0.0	Fraction ET from Active GW Outflow
AGWETP	-	0.6	0.6	Fraction ET from Active GW Storage
LZETP	-	0.4	0.4	Lower Zone ET Parameter
Lateral Transport of Water:				
LSUR	feet	6880	28960	Length of Overland flow Plane
SLSUR	-	0.0007	0.0140	Average Overland Flow Plane Slope
NSUR	-	0.25	0.25	Manning's n For Overland Flow

TABLE 2. MEASURED AND SIMULATED VOLUMES OF STREAMFLOW, REACH 1

Water Year	Month	Measured Flow Acre-ft	Simulated Flow Acre-ft
1978	Oct.	1,107.0	805.8
	Nov.	84.5	6.7
	Dec.	312.3	44.3
	Jan.	1,063.0	531.1
	Feb.	2,338.0	1,526.6
	Mar.	2,528.0	2,196.2
	Apr.	139.8	77.0
	May	304.9	310.5
	June	223.1	348.2
	July	651.6	1,447.8
	Aug.	2,533.0	3,271.9
	Sept.	239.1	975.7
	Total	11,524.3	11,541.8
1979	Oct.	11.1	384.8
	Nov.	0.0	5.1
	Dec.	0.0	28.8
	Jan.	84.2	734.4
	Feb.	149.4	439.3
	Mar.	670.1	618.0
	Apr.	11.3	0.9
	May	2,742.0	2,439.4
	June	89.2	586.0
	July	18.4	113.8
	Aug.	3,418.0	2,943.3
	Sept.	7,793.0	5,330.4
	Total	14,986.7	13,624.2
1980	Oct.	4,217.0	4,693.3
	Nov.	562.8	1,598.8
	Dec.	377.4	593.8
	Jan.	369.5	92.9
	Feb.	300.2	26.4
	Mar.	435.2	42.2
	Apr.	339.4	21.2
	May	99.0	55.4
	June	101.1	8.0
	July	799.1	436.7
	Aug.	779.6	669.3
	Sept.	94.6	187.1
	Total	8,474.9	8,425.1

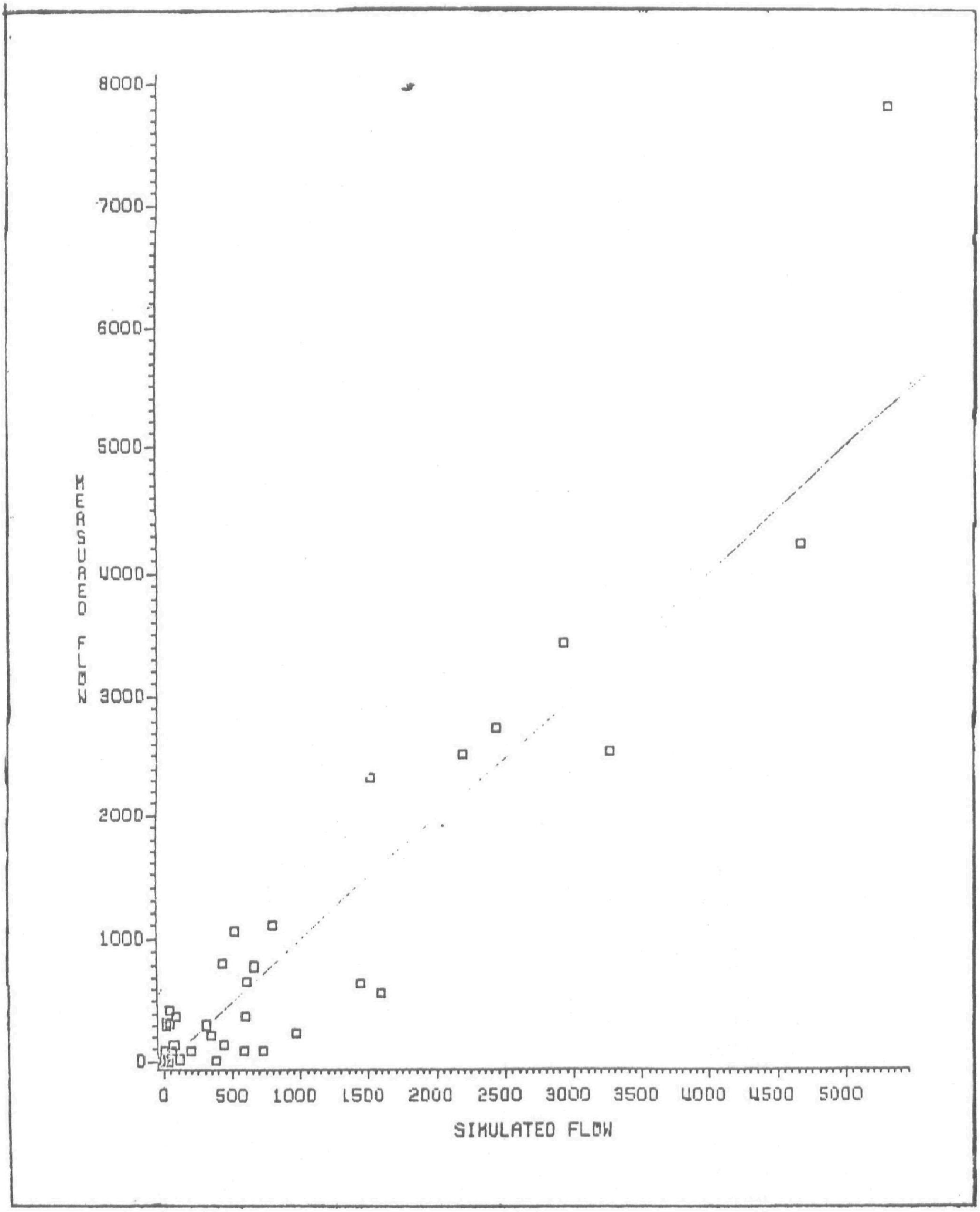


Figure 3. Scattergram for 3 years of monthly measured and simulated flows, volumes in acre-ft.

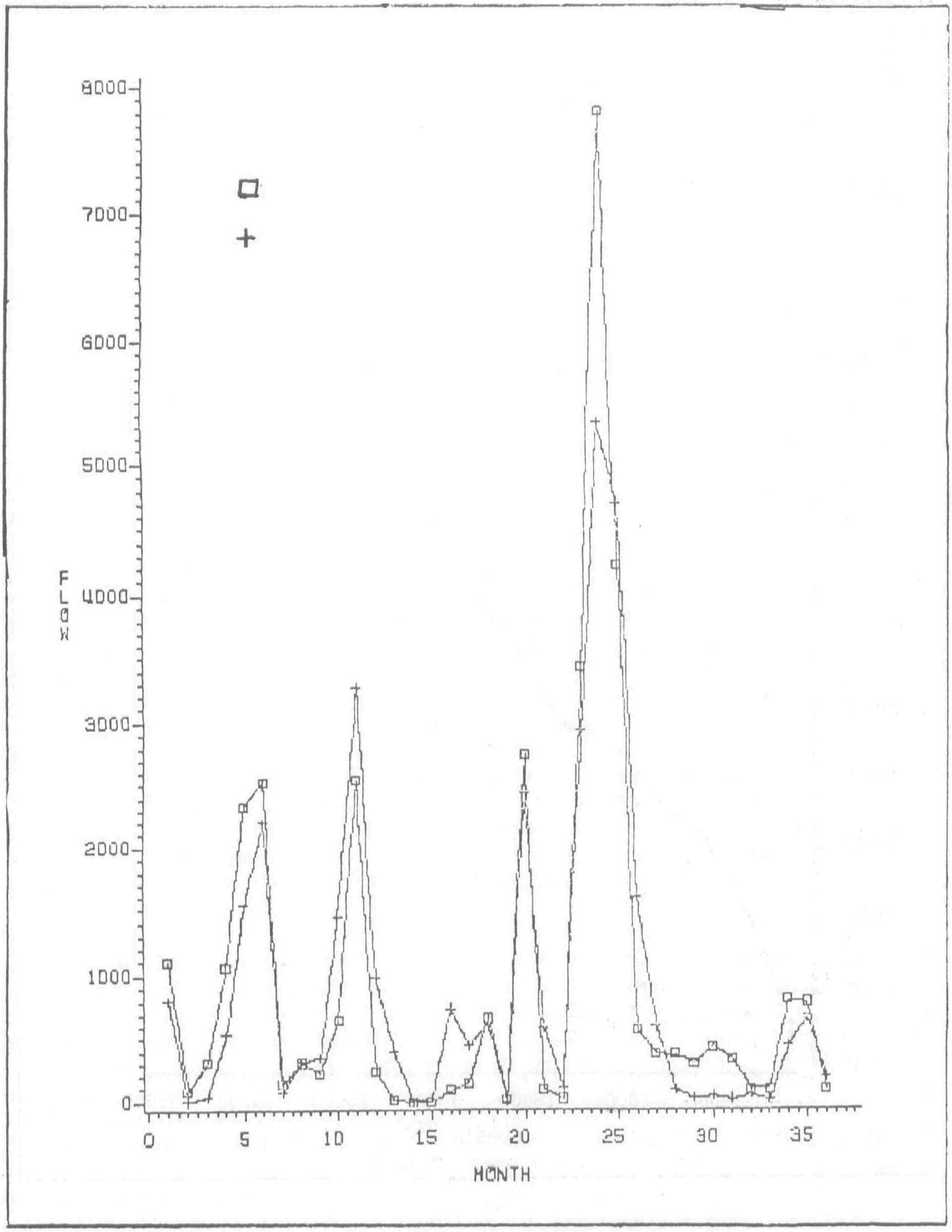


Figure 4. Hydrograph of measured and simulated flows, volumes in acre-ft.

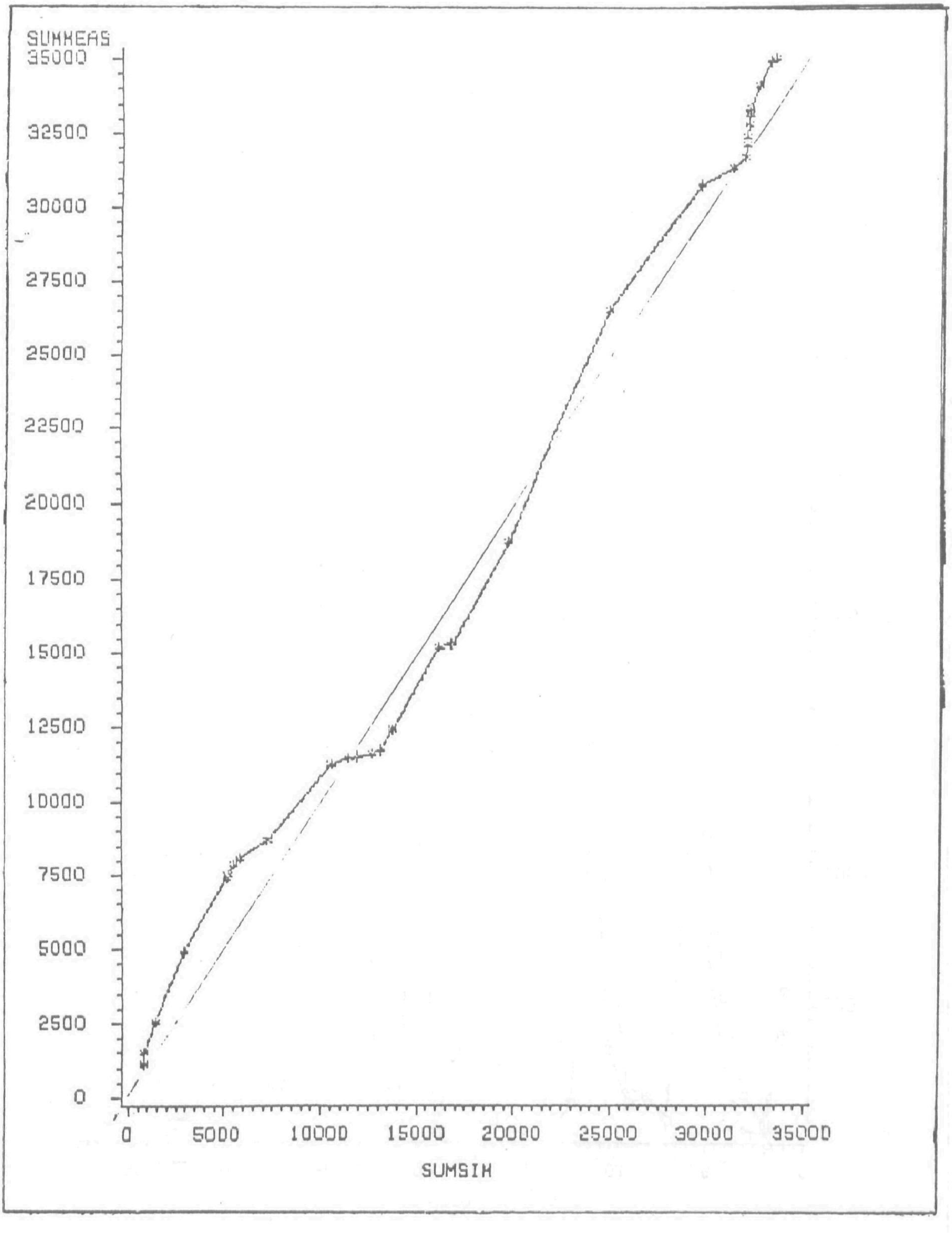


Figure 5. Double mass curve of 3 years of measured and simulated monthly flows, volumes in acre-ft.

DEEP AQUIFER PUMPING

Any water leaving active groundwater storage by percolation into deep groundwater is lost to the HSPF simulation. HSPF cannot add or subtract specific water volumes directly from a deep aquifer. The parameter which controls the rate at which water leaks from the surficial to the deep aquifer is called DEEPFR; DEEPFR is entered as a fraction between 0 and 1.0 and governs the percentage of water entering the active groundwater storage and leaving this block via deep percolation. One possible effect of deep aquifer pumping might be to increase this leakance rate through the semi-confining layer.

This study has broached three methods to capture this possible effect. The first method is to calibrate the model during years when the wellfield was in operation, estimating the appropriate leakance rate, DEEPFR, for this period. Verification of the simulation during years prior to wellfield operation would either confirm that no change in the leakance rate had taken place or suggest that the rate DEEPFR should be lowered for the pre-pumping period.

The second method would involve the use of a continuous lateral outflow from active groundwater storage using a time series. HSPF has available a time series, AGWLI, which can be used to input a continuous inflow into the surficial aquifer; the substitution of negative numbers can change this time series to an outflow. A percentage of the daily pumpage, transformed into units of inches per day, would be used in this time series to mimic the possible increased leakance of water into the deep aquifer, over and above the normal recharge rate.

A third possibility involves the coupling of the HSPF surface hydrologic simulation with a groundwater model. The Prickett model of the Cypress Creek basin used by SWFWMD would be used to generate numbers for HSPF, either a leakance rate to input as DEEPFR or a daily groundwater storage loss which could be input using the AGWLI time series. A quasi-three-dimensional groundwater model of ten municipal wellfields including Cypress Creek estimated that pumping 133 mgd increased leakage to about 9 inches per year (Hutchinson, 1984). A digital model of predevelopment flow by Ryder (1982) estimated a downward leakage of 5 inches per year with no pumping.

CONCLUSION

The number of parameters involved in hydrological simulation using HSPF do not readily lend themselves to physical interpretations. All available data should be used to estimate values for parameters, such as nominal soil storage capacities, but these values should be viewed for the most part as indicators, subject to change during calibration. With this in mind, HSPF can be used with some success to simulate the peculiar conditions of Florida watersheds.

Simultaneous simulation of surface and groundwater flows must be accomplished using indirect methods. The parameter DEEPFR or the time series AGWLI are available in HSPF; consideration should be given to linkage of these values with output from a groundwater model of the watershed.

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HYDRODYNAMIC AND WATER QUALITY SIMULATIONS IN AN ESTUARY WITH MULTIPLE OCEAN BOUNDARIES

by: Ivan B. Chou, P.E.
Applied Technology and Management, Inc.
Gainesville, Florida 32607
and Larry J. Danek, Ph.D.
ESE, Inc.
Gainesville, Florida 32602

ABSTRACT

RECEIV-II model was used to assess the hydraulic and water quality impacts on Pungo River, North Carolina due to a proposed facility in the Albermarle-Pamlico Peninsula. The study area is hydraulically connected with the Atlantic Ocean by both the Albermarle Sound at the north and the Pamlico Sound at the south. The quantity module of the model was calibrated and verified by the results of a dye study data, 5 tide gages, 2 *in situ* current meters, and the wind, precipitation and flow data. The quality module of the RECEIV-II was calibrated and verified by two years of salinity, DO, and BOD measurements at 12 water quality stations. The temperature, solar radiation, chlorophyll *a*, and secci depth data were used to establish the photosynthesis and respiration rate constants. The benthic oxygen demand was also measured on site.

The effects of wind, tide, and plant discharge were assessed by the calibrated model. It was found that under low flow conditions, the tide range was the most sensitive hydraulic parameter in water quality predictions. Also, the wind direction, under certain circumstances, could have significant effects on the water quality in a multiple ocean boundary estuarine system. The wind induced setup will cause net circulation in the estuary and consequently increase the ocean exchange.

INTRODUCTION

North Carolina has an estimated 1,000 square miles of peatland containing about 600 million tons of moisture-free peat. The largest deposits are located on the Albemarle Pamlico Peninsula, which contains an estimated 360 square miles of peatland and 210 million tons of moisture-free peat. These large, concentrated deposits have made the Peninsula an area of primary interest for peat production.

Peat Methanol Associates proposed a synthetic fuels project on the eastern peninsula of North Carolina near Creswell. The development would include the construction of a peat-to-methanol plant. To evaluate the potential environmental impacts of the plant discharge, a comprehensive environmental study was conducted between October 1981 and December 1983 to establish the baseline information for hydrology, water quality, terrestrial and aquatic ecology in the study area, and to predict the hydrological and water quality impacts caused by the proposed facilities.

The hydrological and water quality impacts were assessed by model simulation using RECEIV-II model.

STUDY AREA

The Albemarle-Pamlico Peninsula (Figure 1), located in the tidewater region of North Carolina, is bounded on the east by Pamlico and Croatan Sounds, the west by the Suffolk Scarp (an ancient marine terrace), the north by Albemarle Sound, and on the south by the Pamlico River. The study area and proposed plant site are located near the center of the peninsula within the region of peat deposits. Operations to develop the area began as early as 1787, with the completion of a canal that drained 10,000 acres for farming. Drainage and development projects continued in the area with completion by 1843 of New Lake Canal and Pungo Lake Canal. By the late 1800s and early 1900s, the area was a thriving farming and lumbering community. It was not until the early 1970s, however, that the extensive canals and drainage systems that exist today were developed. Drainage and subsequent land clearing have opened the area for large-scale farming and livestock operations.

BASELINE DATA

Intensive meteorological, hydrological, and water quality measurements were conducted. Figure 2 shows the locations of the water quality and gaging stations. The water quality model was calibrated primarily by three sets of data: November 1981, April 1982, and August 1982.

METEOROLOGY

Historical and field meteorological data were collected and analyzed to characterize the meteorological conditions near the site and support hydrology and water quality modeling. Historical long-term meteorological data from the

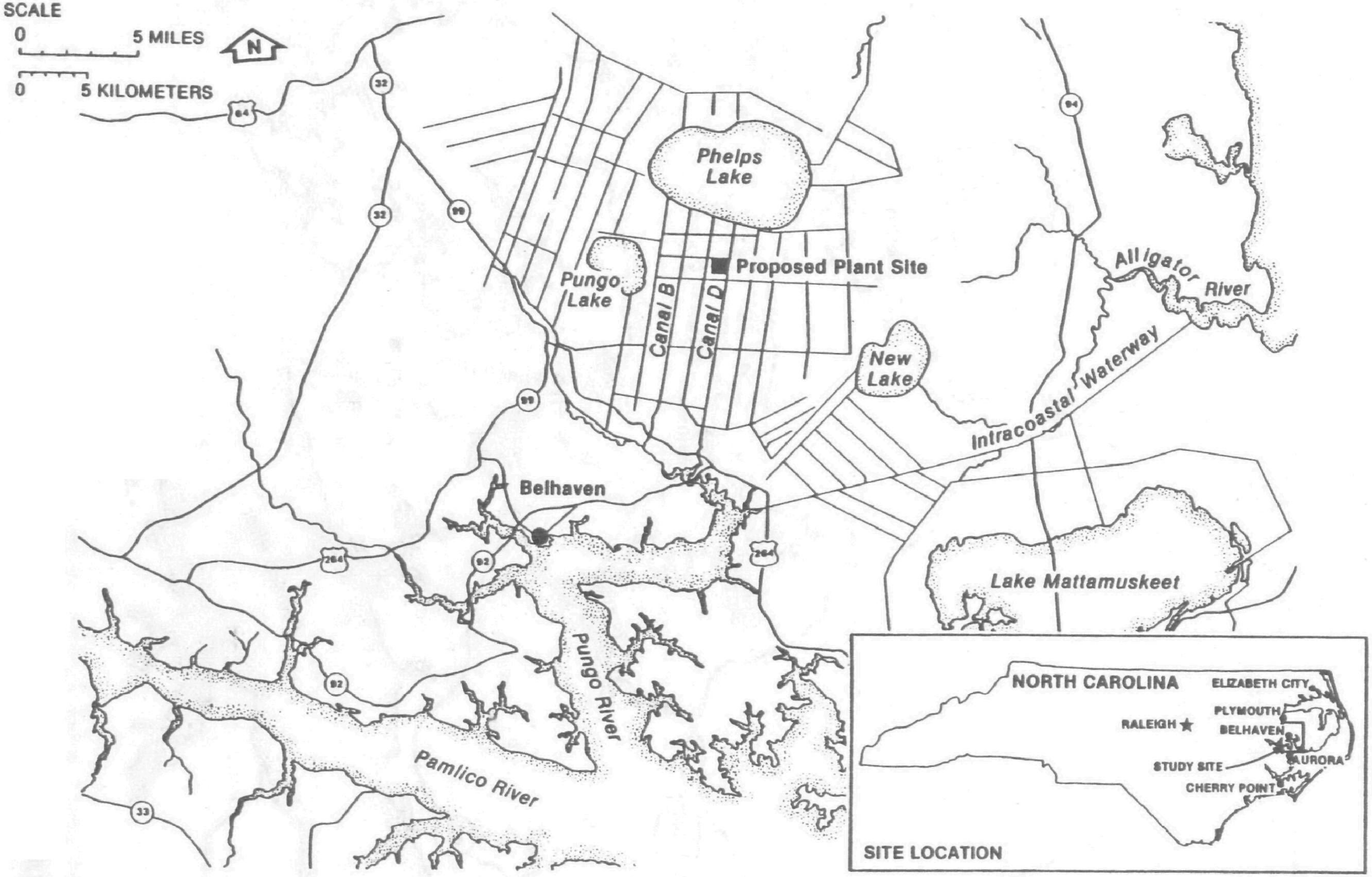


Figure 1
PROJECT LOCATION MAP

SOURCES: USGS, 1980; ESE, 1982.

Figure 2
WATER QUALITY SAMPLING STATIONS AND GAGING STATIONS

SOURCES: USGS, 1980; ESE, 1982.

National Climatic Center were available at four stations (Cherry Point, Elizabeth City, Belhaven, and Aurora, North Carolina) in the study site vicinity. Approximately 5 years of wind, precipitation, evaporation, solar radiation, and temperature data were provided by First Colony Farms. An on-site meteorological station was also maintained during the study period. Around the study site winds were generally from the southwest or north-northeast, usually at speeds less than 17 knots; mean annual precipitation was approximately 50 inches with a maximum and minimum of 75 and 35 inches, respectively; monthly evaporation rates ranged from 2.5 to 11.0 inches; and mean temperature was approximately 65°F, with a maximum of 105°F and a minimum of 5°F.

BATHYMETRY

A bathymetric survey of the Pungo River and adjacent tributaries was conducted to obtain cross-sectional areas at approximately 0.5-mile intervals, including thirty-three transects. The width of the river ranged from 66 feet (ft) in the Pungo River Canal to 1,850 ft at the lower end of the survey area. Average depths ranged from 5.9 to 11.6 ft.

TIDES

Site-specific tide data were collected at four tide gages for up to 16 days during the November 1981 survey at Stations B1, P1, CMC2, and CD3. During the August 1982 survey, three tide gages were operated for 1 month at Stations B1, P1, and CB9. Figure 2 indicates the stations where tide gages were installed. The data from these tide gages were used to calibrate the hydraulic model.

National Oceanic and Atmospheric Administration's Tide and Tidal Current Tables for the eastern coast were examined to obtain historical tide data for the study site. In addition, the U.S. Army Corps of Engineers (COE) maintained 24 years of data at the Belhaven tide gage (1957-1982). The COE data were used to obtain such statistical parameters as mean sea level, mean high water, and mean low water. The long-term average of the tide range at Belhaven (0.5 feet) was used as the tidal boundary condition for model simulation.

Wind set-up effect was noticeable on the water level data. The winds had two distinct effects on the tides. The first was regional winds acting generally on the Pamlico Estuary causing long-period variations in tides. The second wind effect was noticeable but of lesser magnitude. This effect was due to local winds producing unique short-term variations. These variations were superimposed on the lunar tides and, with sufficiently strong winds, would nearly obscure the tidal effect.

CURRENTS AND FLOWS

Two *in situ*, continuous recording current meters were installed near stations P2 and P6 (Figure 2) and several instantaneous measurements were made with an electromagnetic current meter at selected locations throughout the study area. These current data were used to verify the hydraulic model. A weir was installed in Canal B for a 1-month period to monitor flow. Also, three types of dye studies were used to determine the flow conditions: (1) an 8-day continuous injection (dilution study) was conducted in Clark Mill Creek; (2) a time-of-travel study was conducted on Canal B; and (3) net-drift studies were conducted at Stations P1 and P3. The dye studies indicated the net flow issuing from Clark Mill Creek was about 6.0 cfs or approximately 0.2 cfs per square mile (mi^2) of drainage area during the November study period.

HYDROLOGY

To compute the hydrologic parameters for the drainage basins in the study area, the base flow values computed from the available USGS data from the two nearby basins were used: Herring Run near Washington, NC (16-year record) and Albemarle Canal near Swindell, NC (3-year record). Each of the drainage areas has topography, vegetation, land use, and soil cover similar to the study area. Each of the drainage areas contains numerous man-made canals which provide drainage and which are similar to canals found in the study area. Hydrologic characteristics of both watersheds are presented in Table 1. The Log-Pearson Type II distribution technique was used to compute the base flow statistics from the Herring Run data under 7Q10, 30Q2, and IQ2 conditions. Only the low-flow discharges for the 1Q2 flow regime could be calculated for Albemarle Canal since the period of record was not sufficient to calculate the 7Q10 or 30Q2 flows. The calculated base flow during low-flow conditions compares closely with the 1Q2 base flow for Herring Run and Albemarle Canal (0.053 versus 0.051 cfs/mi^2). It was assumed that the statistical analysis of Herring Run data could be applied directly to the study site.

TABLE 1. HYDROLOGIC CHARACTERISTICS OF COMPARISON WATERSHEDS

	Herring Run	Albemarle Canal
Drainage Area:	15 mi^2	68 mi^2
Period of Record:	16 years (1965-1981)	3 years (1977-1981)
Average Flow:	10.5 cfs	102 cfs
Low Flows		
7Q10	0.7 cfs	--
30Q2	1.1 cfs	--
1Q2	0.8 cfs	3.5 cfs
Base Flows		
7Q10	0.047 cfs/mi^2	--
30Q2	0.073 cfs/mi^2	--
IQ2	0.053 cfs/mi^2	0.051 cfs/mi^2

Sources: USGS, 1981; ESE, 1982.

WATER QUALITY

Water quality sampling of the Pungo River was conducted in three field trips conducted on November 4, 1981; Aril 21, 1982; and August 11, 1982. The water quality sampling stations are shown in Figure 2. During the November 19, 1981 and August 11, 1982 sampling trips, morning and afternoon samples were collected from each station. The water quality parameters include: TSS, dissolved solids, turbidity, alkilinity, pH, DO, BOD, CBOD, chlorophyll a, 7 dissolved ions, 9 nutrient parameters, and 18 metals.

Sediment Oxygen Demand

On August 12-13, 1982, *in situ* SOD tests were conducted using benthic chambers. The chambers were designed to minimize the volume-to-surface area ratio. Both light and dark chambers were used in order to separate SOD and photosynthesis. SOD measurements were conducted at Stations P4, P9, and P11. Incubation times ranged from 2.5 hours to 5 hours. The SOD rate was computed by first subtracting out the water column BOD oxidation, and then temperature correcting to an equivalent 20°C rate. The SOD rates at 20°C range from 0.03 $gm/m^2/day$ to 0.19 $gm/m^2/d$. SOD values at these stations are lower than those reported by Pomeroy and Wiegert (1981) (1) for estuarine sediments and also lower than values for lacustrine sediments, compiled by Belanger (1981) (2).

Primary Productivity

Net algal photosynthetic oxygen production can be a significant source of dissolved oxygen in a system. Estimates of algal photosynthesis and respiration (P-R) were made using the November 1981 and August 1982 Pungo River chlorophyll a, temperature, solar radiation, and secchi depth data, and generally accepted phytoplankton kinetic rates.

The calculations of algal photosynthesis involved the application of a light attenuation factor to an ideal algal growth rate and then calculating the amount of oxygen produced during algal carbon fixation. The light extinction coefficient was determined from secchi depth measurements. Net algal oxygen production was greater in the downstream reach in August 1982 and in the upstream reach in November 1981. These results correspond to the reach which had the greater phytoplankton biomass (as indicated by chlorophyll a) for both surveys.

Wastewater Loads

There were eight NPDES permitted dischargers within the model segmentation. The major discharger is the Belhaven WWTP which was permitted to have an effluent of 0.5 MGD with 125.0 lb/day monthly average BOD_5-20°C loading. The only discharge reports relevant to BOD/DO calibration were from Belhaven WWTP which reported to have had a BOD_5-20°C loading of 94.0 lb/day on November 11, 1981, and 81.9 lb/day on August 10, 1982. Since there was no other discharge information available during the field sampling period, the BOD loadings were assumed to be the discharge limitation stated in the NPDES permits.

MODEL DESCRIPTION AND CALIBRATION

MODEL DESCRIPTION

RECEIV-II is a dynamic, link-node hydrodynamic and water quality model. The governing equation and numerical scheme is one-dimensional in a strict sense; however, the model nodes can be linked to construct a two-dimensional network and can simulate a two-dimensional, vertically integrated system. An important feature of the model is that it can simulate multiple ocean boundary conditions. These multiple boundary conditions had to be considered in Pungo River modeling because the Pungo River is connected to the Albemarle Sound by the Intracoastal Waterway and the Alligator River. The interaction of the tidal forcing at Albemarle Sound and Pamilico sound have dynamic effects on the circulation and tidal flushing.

The quality block of RECEIV-II can simulate the phosphorus cycle, nitrogen cycle, coliform, chlorophyll a, BOD/DO, salinity, and metal ion concentrations. Although abundant nutrient and chlorophyll a data were collected during the course of the study, only BOD/DO, salinity, and metal ion were considered because of the high confidence level to be expected. Instead of solving the coupled equations for nitrate, nitrite, ammonia, phosphorus, chlorophyll a, and oxygen, the net P-R was estimated by RPW (1982)(3) using the previously described method. The model input of SOD rates was adjusted to include the P-R effects using the following formula:

$$SOD' = SOD - (P\text{-}R) \times D$$

where SOD' = adjusted SOD, and
D = water depth.

The RECEIV-II model uses Churchill's (1962)(4) formula to compute reaerations rate (K_a). This method, however, could underestimate the reaeration under low velocity conditions. Therefore, the model was modified to impose a lower limit, $K_a \geq 2/D$, suggested by O'Conner (1978)(5), for reaeration coefficient, where K_a is the reaeration rate in day^{-1}, and D is the water depth in feet.

The quantity block of the RECEIV-II model was calibrated using the tide, current, flow, and wind data collected in August 1982 and was verified further using the November 1981 hydrologic and meteorological data. The quality block of the RECEIV-II model was calibrated using chloride, BOD, and DO data collected in August 1982, and was verified further using the November 1981 chloride, BOD, and DO data. A dye study conducted in November 1981 was used to determine the freshwater discharge and to calibrate the quality block of the model.

Several parameters need to be determined by model calibration. These are:

1. Manning's roughness coefficients (n),
2. ocean exchange coefficients,
3. BOD deoxygenation rate,
4. SOD, and
5. net photosynthesis and respiration (P-R) rate.

MODEL SEGMENTATION

RECEIV-II model was set up for the Pungo River system using 48 nodes and 49 channels. The segmented model included the Pungo River Canal, Pungo River, tidally influenced portions of Canal B and Canal D, Clark Mill Creek, the Intracoastal Waterway and portions of the Alligator River. Figure 3 shows the locations of all model nodes. The shortest segment in the system was 2000 feet, which prompted choosing a hydraulic time step of 90 seconds to maintain numerical stability.

HYDRODYNAMIC CALIBRATION

The RECEIV-II model is a quasi-steady dynamic model which assumes the tide to be periodic and can accept a maximum of three tidal frequency components. (The term "quasi-steady" is defined as a periodic time varying condition with constant amplitude and phase lag. The function changes with time, but the harmonic cycle is repeated.) Since the observed tide data at all gages were highly irregular, a piecewise calibration technique was used to simulate the non-periodic wind effects as well as the tidal effects. The tide record used for the 1982 calibration period was from 00:00 hr on August 29 to 06:00 hr on August 31, and it was divided into three segments:

1. 00:00 hr August 29 to 04:00 hr August 30,
2. 16:00 hr August 29 to 16:00 hr August 30, and
3. 08:00 hr August 30 to 06:00 hr August 31.

There was an overlapping period between segments.

Following the "cold starting" with arbitrary initial conditions, the water level at nodes and current velocities at channels at the 16th hour of the first calibration segment were used as initial conditions for the second calibration period. Similarly, the water levels and velocities at the 16th hour of the second calibration segment were used as initial conditions for the third calibration segment. Different wind conditions were used for each time segment. This restarting method normally causes some discontinuity between calibration periods due to numerical truncating error, secondary dynamic effects, and non-harmonic boundary inputs. However, the results in the overlapping periods had shown that the effects of discontinuity were insignificant. The maximum mismatch in the overlapping sections was 0.06 ft. The simulated water levels were compared with the 1982 tide data at Belhaven, P1 and CB9, as shown in Figure 4. The simulated current velocities from the model were compared with current meter data at P2 and P6 as shown in Figure 5.

After the quantity block of the RECEIV-II model was calibrated using August 1982 hydrologic data, the model was verified using November 1981 data. The tide record used for the 1981 calibration period was from 20:00 hr November 2 to 00:00 hr November 4, 1981. The results of the 1981 verification using tide data at Belhaven, P1 and CMC2 are shown in Figure 4.

The results of the hydrodynamic calibration showed that the simulated water level followed closely with the observed tide data at four locations using two independent sets of data. The maximum difference between the model simulation and tide data was about 0.1 foot at P2 and smaller at other locations. The phase of both the simulated tide and current agree closely

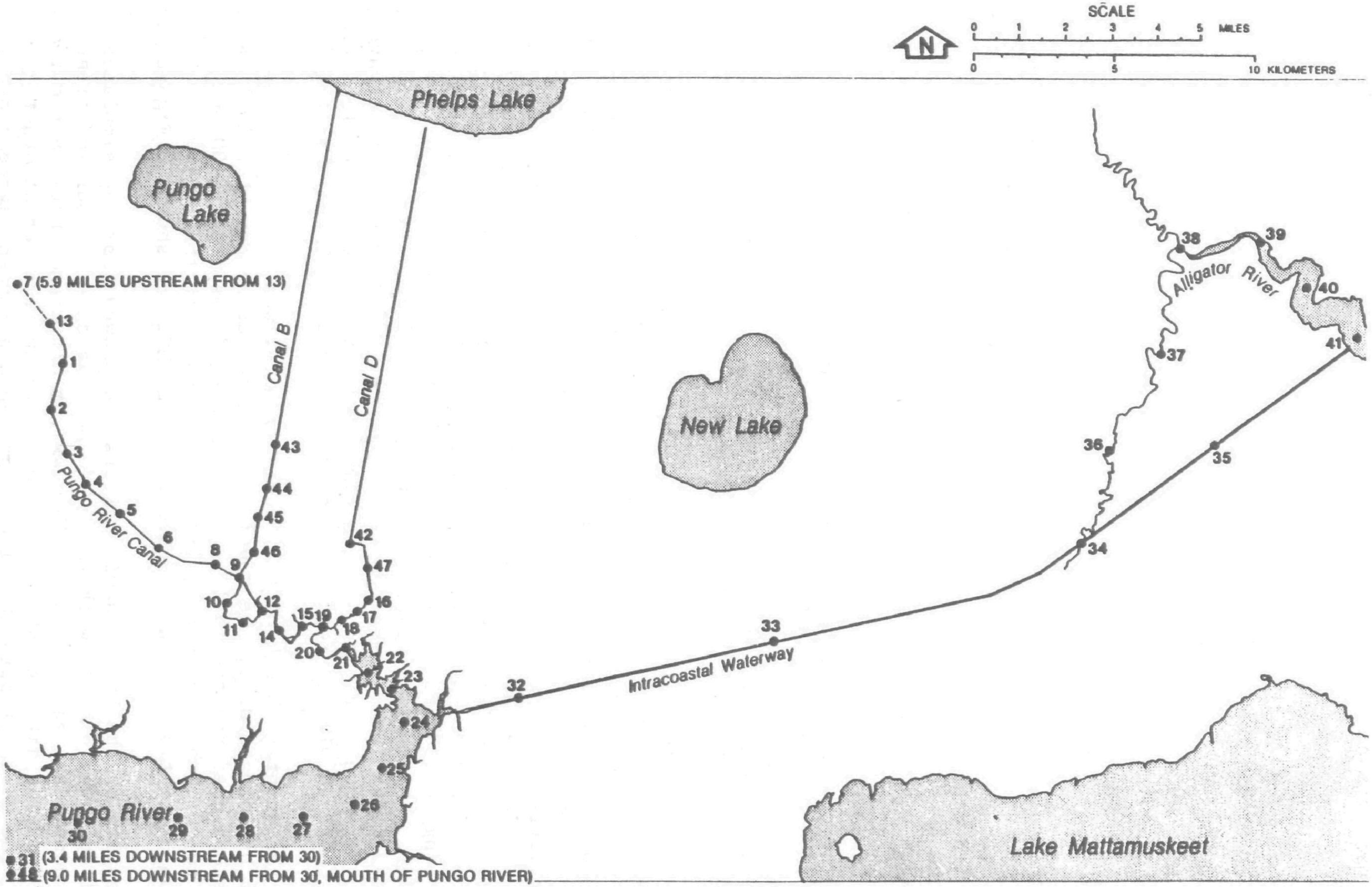
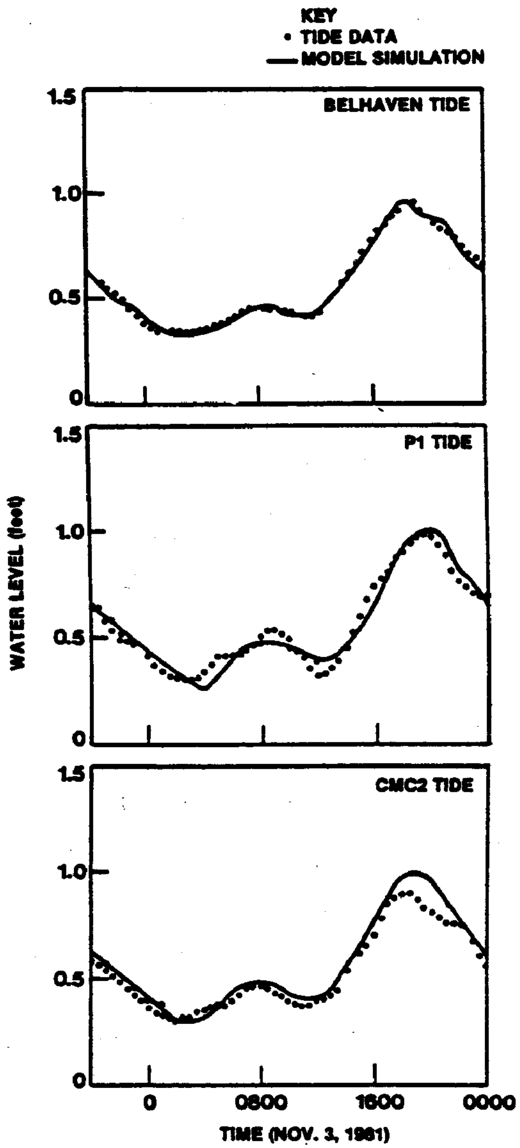
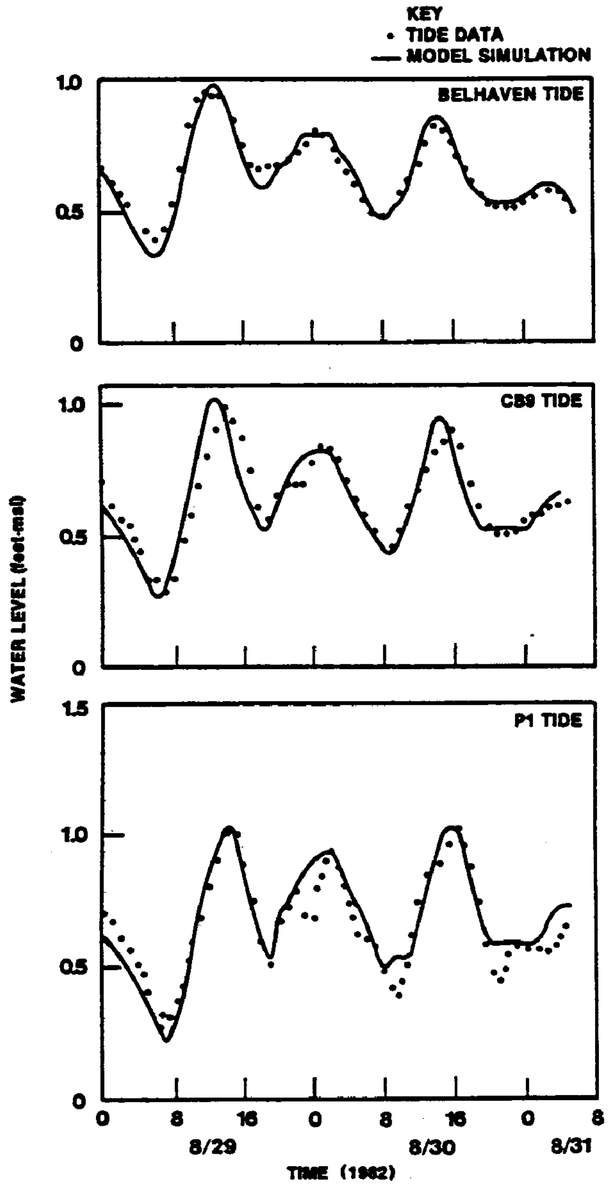


Figure 3
MODEL SEGMENTATION

SOURCE: ESE, 1982.



HYDRODYNAMIC CALIBRATION (1981 TIDE)



(1982 TIDE)

Figure 4
HYDRODYNAMIC CALIBRATION AND VERIFICATION

SOURCE: ESE, 1982.

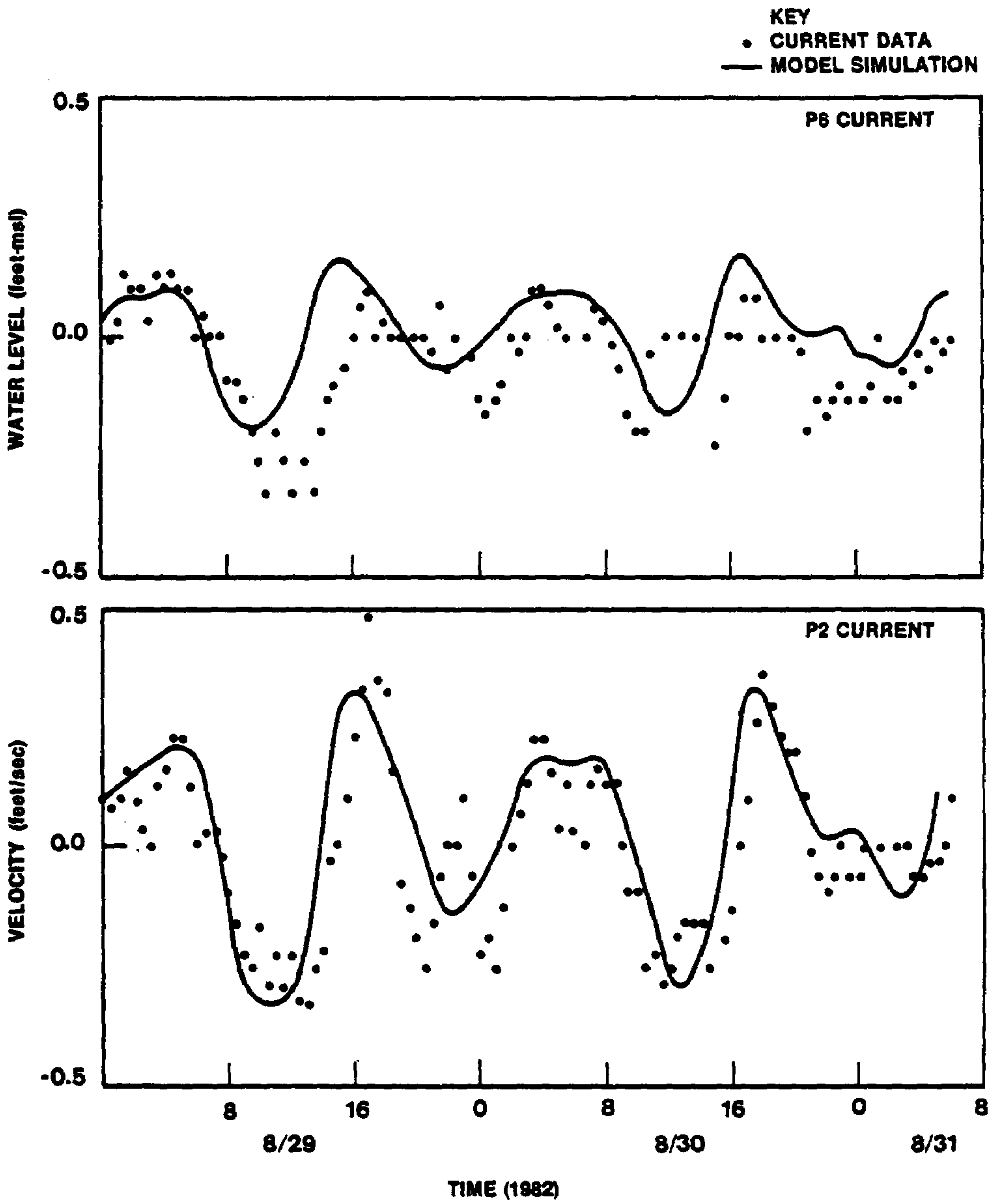


Figure 5
HYDRODYNAMIC CALIBRATION (1982 CURRENT)

SOURCE: ESE, 1982.

with the data. The error of simulated current velocity was generally larger than tide because the current meter was located at a single point in channel cross section, while the simulated current was a cross-sectional average.

The calibrated Manning's n ranges from 0.027 to 0.035.

Dye Calibration

A dye study was conducted by ESE between November 6 and November 14, 1981. The purposes of this dye study were to determine the freshwater discharge from Canal D and Clark Mill Creek and to ensure that the model could properly simulate the transport of the discharge plume in the Pungo River System.

Dye injection with an initial concentration of 4.6×10^7 ppb was begun at 08:41 AM on November 6, 1981, at a rate of approximately 74 ml/min. Automated Isco samplers, indicated that the dye had stabilized approximately 7 days after injection began; consequently, the dye study was begun on the eighth day (November 14, 1981). Dye concentrations were measured at five stations on the Pungo River and Clark Mill Creek as frequently as possible during a tidal cycle. Generally, these stations were sampled once every half hour except when one of the other dye surveys was being conducted. In addition, two longitudinal dye surveys (morning and afternoon of November 14, 1981) were conducted on the Pungo River. The *in situ* photodecay rate of Rhodamin dye was measured to be 0.0172 day^{-1} during the dye study period, and this value was used in the model for calibration.

The results of the dye calibration indicated that the freshwater flow during November 1981 sampling period was 0.22 cfs/mi^2. Figure 6 shows the comparison between mode predicted maximum dye concentration on November 14, 1981 and the measured values.

WATER QUALITY CALIBRATION

The objectives of the salinity calibration were to determine the ocean exchange coefficients at the boundaries, and to verify that the model can adequately simulate the diluting effects from the ocean boundary. The results of chloride calibration showed that the appropriate exchange coefficients were 0.8 at nodes 48 and 0.5 at node 41. Figure 6 also shows the chloride calibration on a long-term average basis and also for dry and wet seasons. The Pungo River system was found to be insensitive to the ocean exchange coefficient especially under low-flow and small tide range conditions.

BOD/DO Calibration

The purpose of BOD/DO calibration is to determine the proper values of the BOD deoxygenation rate (Kd), SOD rate, P-R rate, and distributed BOD loading to be used for RECEIV-II simulation. The water quality data collected in August 1982 were used for calibration, and the November 1981 data were used for model verification. The calibration was an iterative process in which the aforementioned parameters were altered until the model simulation represented a best fit to the observed BOD/DO data. However, the parameter values were

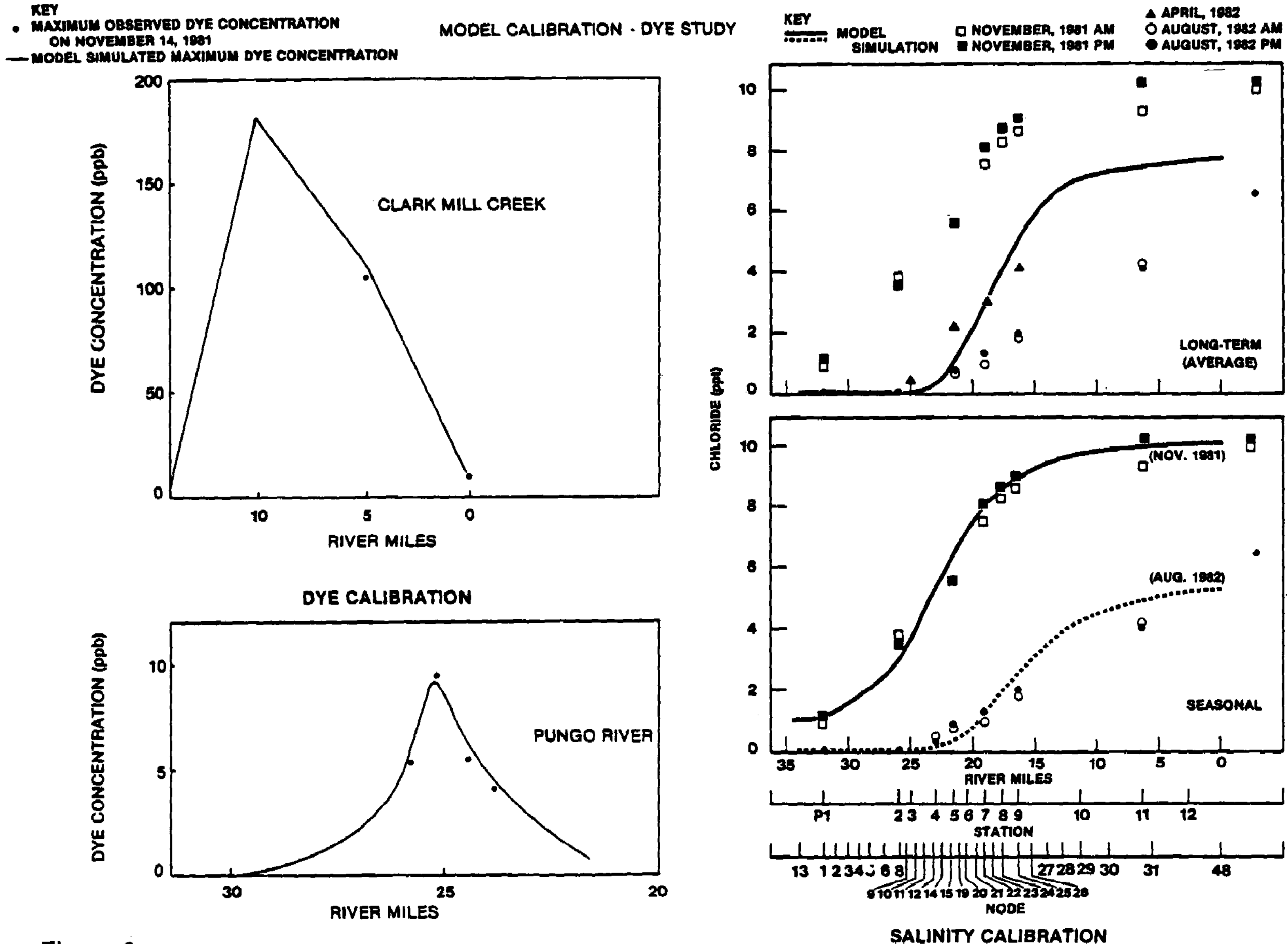


Figure 6
DYE CALIBRATION AND SALINITY CALIBRATION

SOURCE: ESE, 1982.

not determined by curve fitting only; they were also supported by the field data and laboratory analysis.

The carbonaceous BOD (20°C) was measured in the laboratory by incubating the water sample for 50 days. The ultimate carbonaceous BOD and BOD deoxygenation rate (Kd) was determined using Barnwell's (1980)(6) nonlinear least-square method. Figure 7 shows the calculated Kd values at sampling stations.

According to the characteristics of the estuary, the Pungo River was divided into three reaches: reach 1, from river mile 23 to 32; reach 2, from river mile 19 to 23; and reach 3, from river mile 0 to 19. Reaches 1 and 2 were shallow and narrow channels with occasionally turbid water. Reach 3 was deeper, wider, and less turbid. A Kd (20°C) value of 0.15 day^{-1} was used for reach 3, and the Kd (20°C) value of 0.12 day^{-1} was used for reaches 1 and 2. A temperature correction factor of 1.047 was used for Kd.

According to RPW's (1982)(3) analysis of the *in situ* SOD tests, the SOD rate in reach 3 was computed to be 0.14 $gm/m^2/day$ at 29°C. An SOD rate of 1.0 $gm/m^2/day$ was used for reach 2, and an SOD value of 4.0 $gm/m^2/day$ was used for reach 1. The high SOD value was used in the upstream reach because of the effect of swamps in the upper Pungo River and Pungo River Canal. The 1981 DO calibration indicated that a temperature correction factor of 1.08 was appropriate for the SOD rate.

The photosynthesis-respiration rates for August 1982 at 29°C were computed to be 0.63 mg/L/day for reach 3, and 0.21 mg/L/day for reaches 1 and 2. The DO calibration, using November 1981 data, indicated a temperature correction factor of 1.055 could be applied to the P-R rate.

The background BOD loading was assumed to be proportional to the water surface area. The lineal distribution and volumetric distribution of BOD loading methods were also attempted, but the area distribution seemed to function better. The 1981 and 1982 BOD calibrations indicated that the distributed BOD loadings in August 1982 at 29°C were 1.0 $gm/m^2/day$ for reaches 1 and 2 and 2.5 $gm/m^2/day$ for reach 3. A temperature correction factor of 1.059 was used for the BOD distributed loadings. A summary of the rate constants obtained from BOD/DO calibrations is presented in Table 2.

Figure 8 shows the 1981 and 1982 BOD/DO calibration.

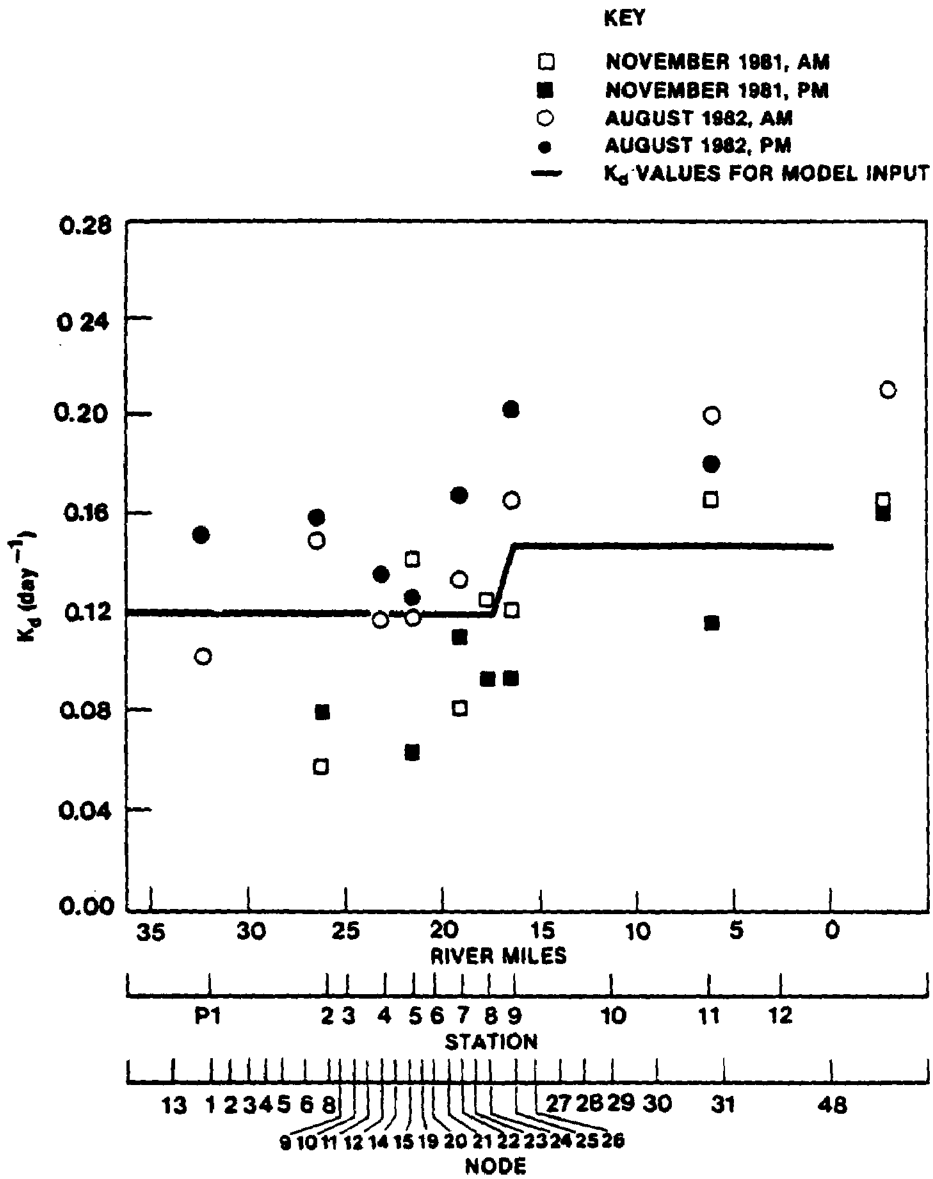
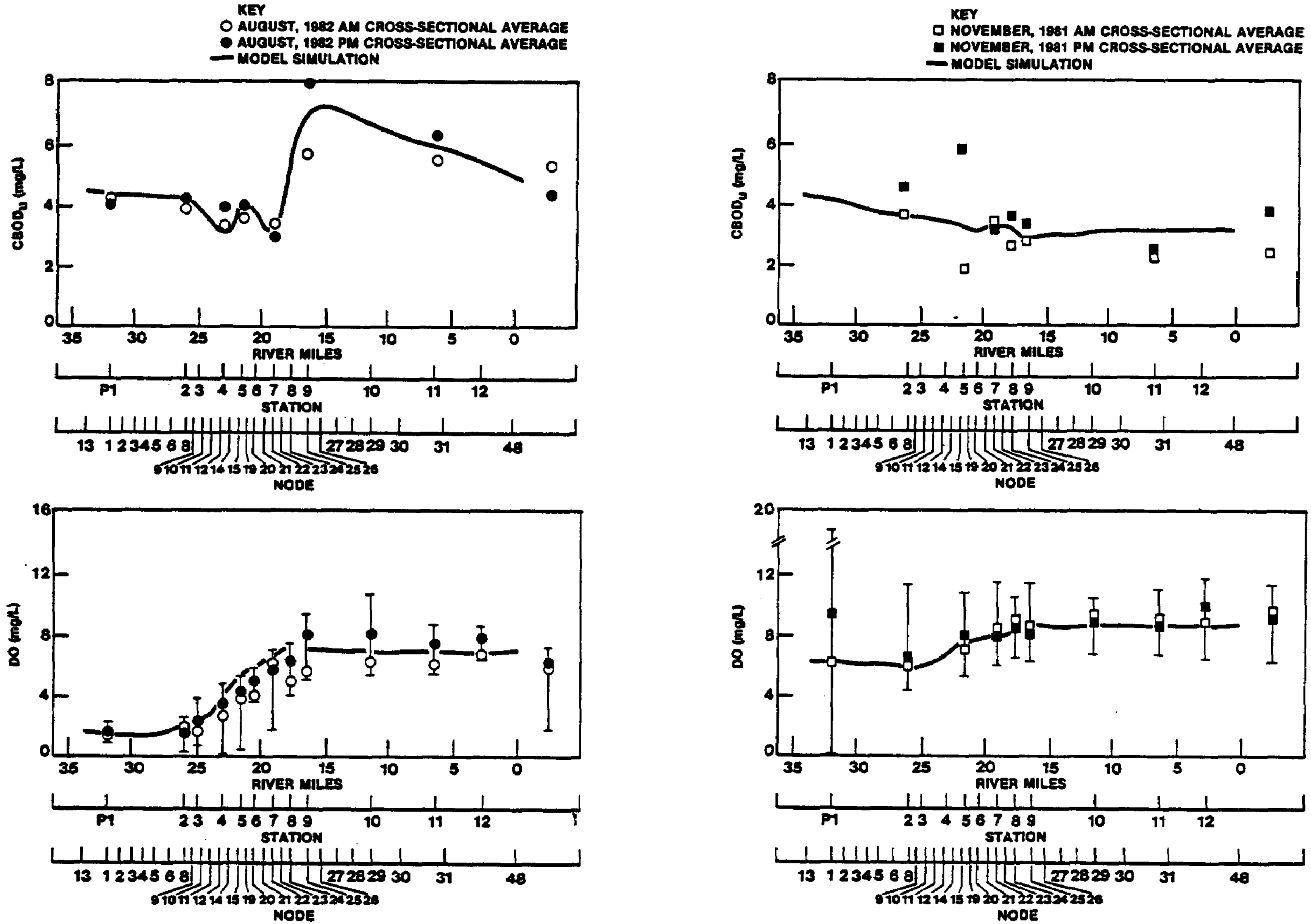


Figure 7
BOD DEOXYGENATION RATE (K_d)

SOURCE: ESE, 1982.



BOD/DO CALIBRATION (1982 DATA)

BOD/DO CALIBRATION (1981 DATA)

Figure 8
BOD/DO CALIBRATION AND VERIFICATION

SOURCE: ESE, 1982.

TABLE 2. RATE CONSTANTS FOR BOD/DO SIMULATION

Parameter	Reach*	Base Rate at 20°C	Temperature Correction Factor	August 1982 (29°C)	November 1981 (17°C)
Distributed BOD loading (gm/m^2/day)	1, 2	0.59	1.059	1.0	0.5
	3	1.55	1.059	2.6	1.3
SOD (gm/m^2/day)	1	2.0	1.08	4.0	1.59
	2	0.5	1.08	1.0	0.4
	3	0.07	1.08	0.14	0.06
P-R (mg/L/day)	1, 2	0.13	1.055	0.21	0.11
	3	0.39	1.055	0.63	0.33
K (day^{-1})	1, 2	0.12	1.047	0.18	0.10
	3	0.15	1.047	0.23	0.13
Reaerations (day^{-1})		**	1.024		

*Reach 1 = River mile 23-32.
Reach 2 = River mile 19-23
Reach 3 = River mile 0-19

** $$K_a = \frac{11.57 \times U^{0.969}}{D^{1.673}}, \text{ and } K_a \geq \frac{2}{D}$$

Where: U = flow velicity (ft/sec), and
D = water depth (ft).

Source: ESE, 1982.

RESULTS

The 7-day-10-year low flow condition was used as the hydraulic input for model projection. Sensitivity analysis was conducted to test the effects of tide range, wind speed and wind direction on the water quality projections. The results showed that, among these three parameters, tide range is the most sensitive one. Under a low flow condition, changes in tide amplitude will cause large changes in tidal prism and consequently the diluting flows. Figure 9 shows the metal ion dilution ratio under neap tide (0.2 ft), average tide (0.5 ft), and spring tide (1.35 ft) conditions.

According to the results of quantity simulation, the southwest wind stress at water surface will cause water level setup in the upper Pungo River. The setup at station P1 is about 0.1 ft. This wind surge will build up a head differential between the two ends of the Introcoastal Waterway, and drive the water in Pungo River through the Intracoastal Waterway toward Albemarle Sound.

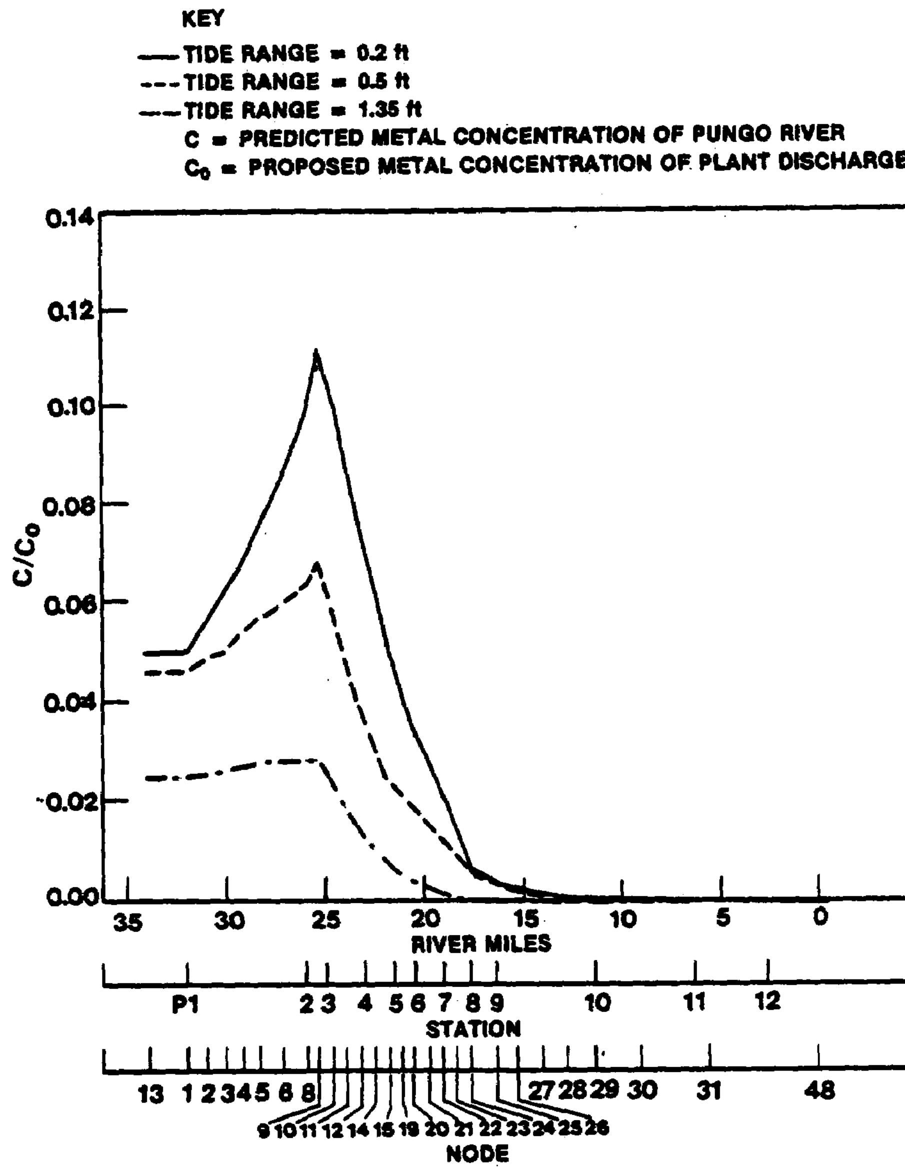


Figure 9
TIDE EFFECTS ON METAL ION CONCENTRATION

SOURCE: ESE, 1982.

The model shows that a southwest wind will cause a flow of 1,560 cfs (0.55 ft/sec) in the Intracoastal Waterway toward Albemarle Sound. Similarly, the northeast wind causes a 0.07-ft water level set-down at station P1 and a 1,180-cfs flow (0.32 ft/sec) in the Intracoastal Waterway toward the Pungo River. This wind induced current in the Intracoastal Waterway was qualitatively verified by field observation. In both cases, the wind will carry the water from Albemarle Sound or Pamlico Sound toward the confluence of the Pungo River and the Intracoastal Waterway (node 24). In effect, node 24 becomes an ocean exchange boundary for the upper Pungo River. Therefore, the wind will increase the dilution ratio in the Pungo River.

Figure 10 shows the effects of wind direction on BOD/DO in the Pungo River under northeast and southwest (the prevailing wind directions) conditions. The results show that the northeast wind caused higher BOD concentration in the Pungo River near the confluence with the Intracoastal Waterway. The northeast wind tends to carry the water from the Intracoastal Waterway, which has a high BOD concentration, into the Pungo River, therefore increasing the BOD near node 24. The simulations of the conservative substances also indicate that the wind from either direction will reduce the pollutant concentration in the upper Pungo River.

It is evident that the wind direction has important effects on the water quality in a multiple ocean boundary system. The wind set-up is effective in creating net circulation in the estuary while the tidal flushing alone has gradual effects, especially near the upstream portion of the river. The wind induced circulation will shift the influence of the ocean boundary further upstream of the estuary and it will accelerate the tidal exchange.

ACKNOWLEDGEMENT

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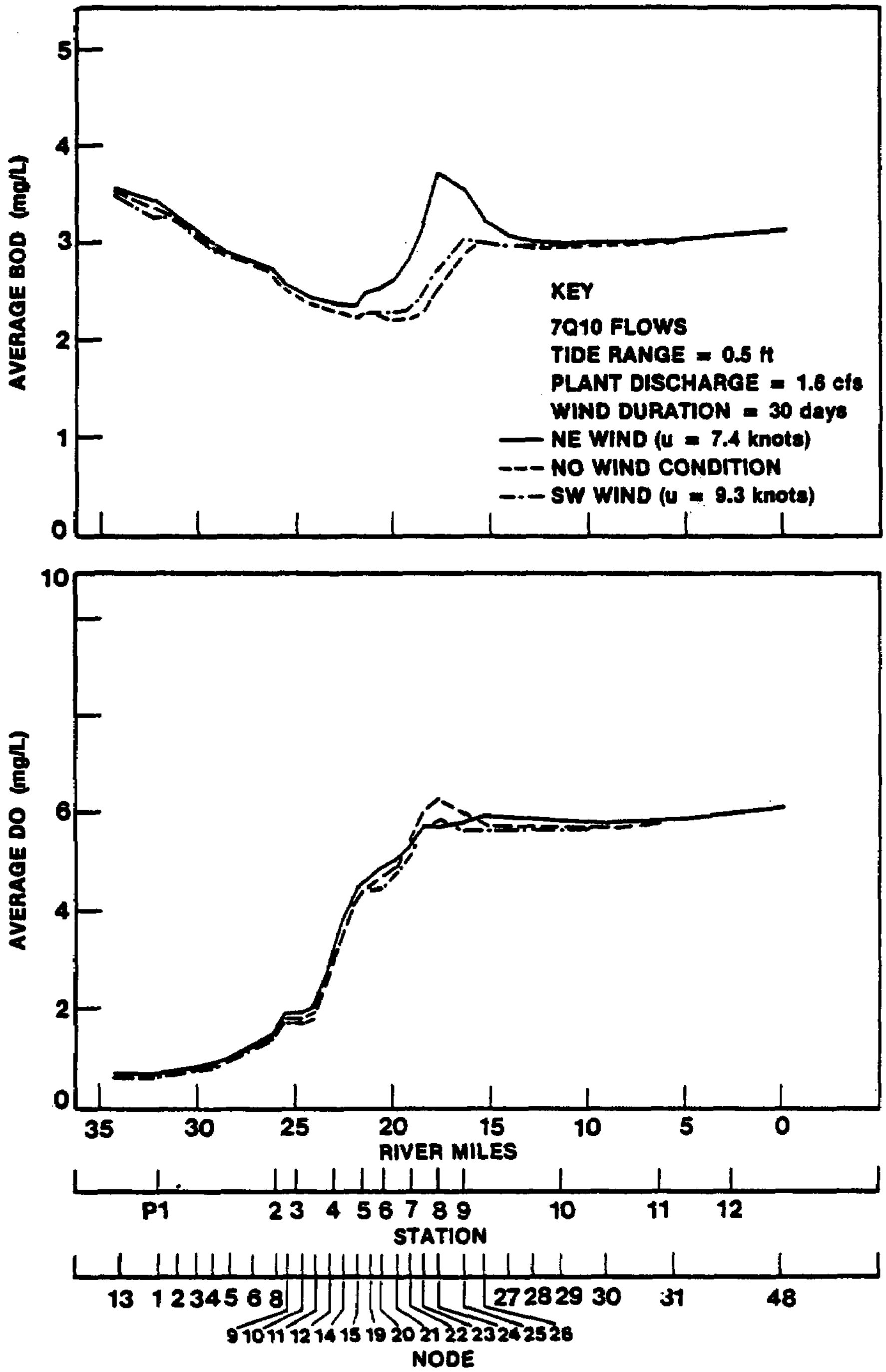

Figure 10
EFFECTS OF WIND DIRECTION ON BOD AND DO

SOURCE: ESE, 1982.

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MODELING ESTUARINE PHYTOPLANKTON-NUTRIENT DYNAMICS USING MICROCOMPUTERS

by: Wu-Seng Lung, PhD, PE
Department of Civil Engineering
University of Virginia
Charlottesville, VA 22901

ABSTRACT

A tidally averaged water quality model was used to study the factors limiting the phytoplankton growth in the James River Estuary in Virginia under dry weather conditions. The water quality parameters calculated by the model are CBOD, dissolved oxygen, organic nitrogen, ammonia nitrogen, nitrite plus nitrate nitrogen, organic phosphorus, inorganic phosphorus, and chlorophyll a. The model incorporates major physical, chemical, and biological processes which link these water quality parameters. The kinetics are modeled in each of the 50 segments in the longitudinal direction of the estuary.

Model calibration analysis using two different data sets collected during the July (low water slack) and September (high water slack), 1983 was very successful. The model reproduces these two data sets using a consistent set of kinetic coefficients. The phytoplankton growth plays a significant role in contributing to ultimate CBOD through respiration of live phytoplankton as well as recycling of carbon from algal biomass and in balancing the dissolves oxygen budget via photosynthesis. More importantly, turbidity plays a key role in limiting the phytoplankton growth and nutrients (nitrogen and phosphorus) are not found to be a key limiting factor in most cases.

The execution of this time variable model on an IBM PC (with 256K RAM) is successful. Further, the use of the 8087 math coprocessor significantly reduces the run time on the PC. Plotting software developed for this study generates report quality plots of data and of model results.

INTRODUCTION

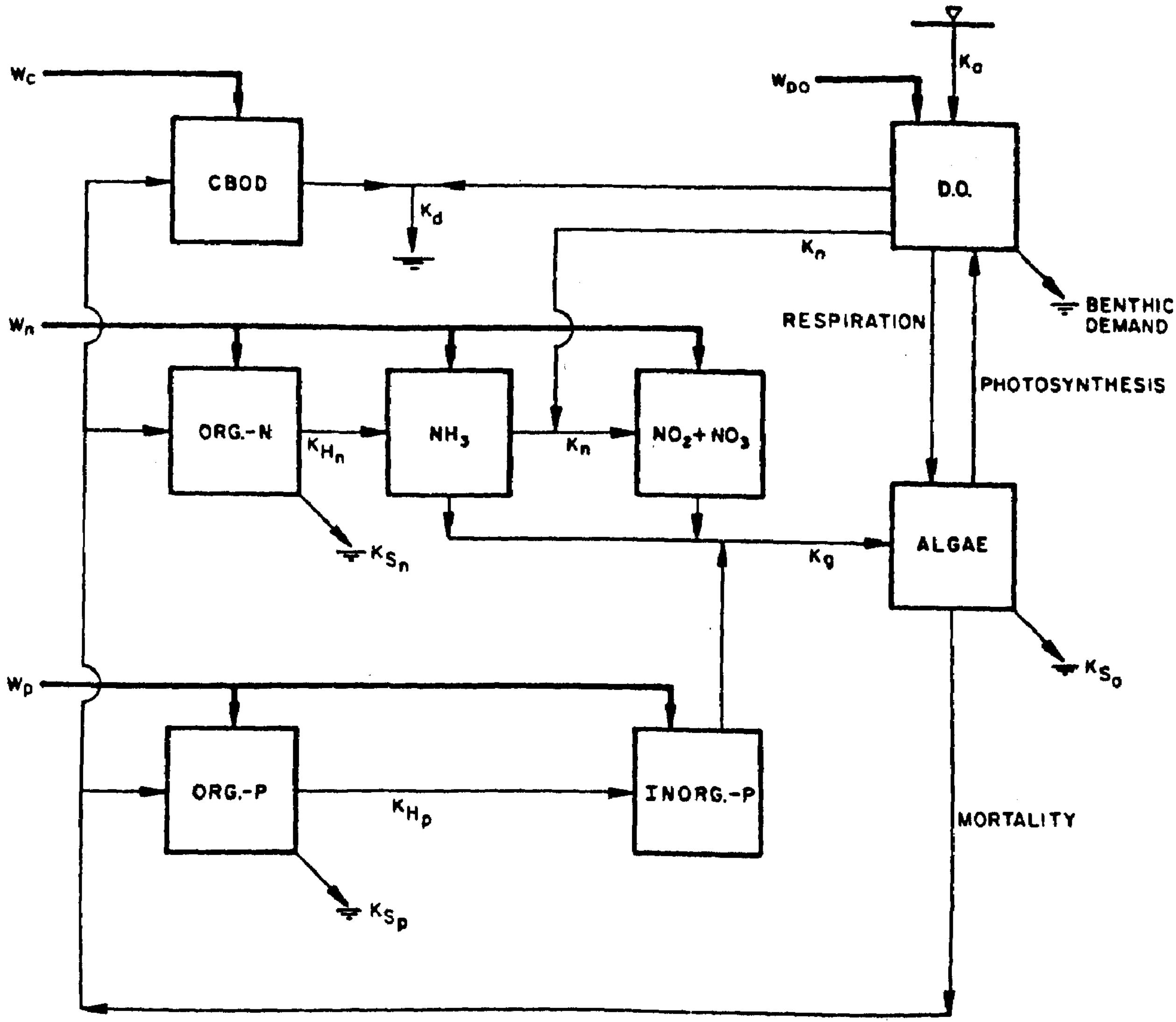
In the past few years, microcomputer systems have become quite powerful in terms of their speed and core memory size that they can handle most of water quality modeling analyses which were once considered not possible on a microcomputer. Parallel advancement in microcomputer software has offered many user-friendly features which significantly improve the efficiency of running some complicated water quality models. Such improvement in both hardware and software already have had considerable impacts on wasteload allocation (WLA) studies as the so-called water quality based approach in water pollution control calls for sharp increase in the use of water quality models in WLA and NPDES permit related studies. Thus, easy access to computing facility and the associated user friendly software is important to the success of many WLA and other water quality modeling studies. To date, many large modeling codes such as WASP (Water Quality Analysis and Simulation Program) have already been installed and run on microcomputer systems successfully.

This study employed an existing water quality model of the upper James River Estuary in Virginia to assess the water quality impacts of potential point source phosphorus control programs in the James River basin (3). This paper presents the results of model calibration and sensitivity analyses using the most recent data currently available on the James. An important feature of this modeling analysis is that the entire modeling analysis was carried out on a microcomputer system, ranging from data plotting, model running, model result printing and plotting. Using such a microcomputer system proves to be much more efficient than using many mainframe computer systems. In addition, the microcomputer system used in this study has very common configuration and a quite modest price tag.

APPROACH AND METHODOLOGY

The James River Estuary model (JMSRV), which was developed by Hydroscience, Inc. (1) for the Virginia State Water Control Board (SWCB), was used in this modeling study. The model was originally intended for use in wasteload allocations of BOD loads from municipalities and industries to meet the dissolved oxygen (DO) standard in the James. The current version installed on a COMPAQ microcomputer is a 50-segment one-dimensional tidally averaged model. [See Figure 1 for the model segmentation.] The kinetic structure of the model is shown in Figure 2. In addition to the BOD/DO kinetics, phytoplankton biomass/nutrient dynamics is also incorporated in the model. As such, the model can be used, in a first-cut analysis, to assess the eutrophication potential and the impact of point source phosphorus load reduction.

The JMSRV model was originally calibrated using the data collected in 1976 and 1978 (1). Although the water quality problem of concern at that time was dissolved oxygen and the emphasis of the modeling analysis therefore was on the verification of the BOD and DO concentrations, the modeling analysis also calibrated the kinetic coefficients of phytoplankton-nutrient dynamics in the upper James River Estuary. In this study, these coefficient values were first used in the preliminary model calibration with the most recent data



W_{DO} = DISSOLVED OXYGEN OF WASTEWATER
W_c = CBOD WASTE LOADS
W_n = NITROGEN WASTE LOADS
W_p = PHOSPHORUS WASTE LOADS
K_a = DISSOLVED OXYGEN REAERATION
K_d = DEOXYGENATION COEFFICIENT
K_n = NITRIFICATION COEFFICIENT
K_{H_n} = HYDROLYSIS RATE OF ORGANIC NITROGEN
K_{H_p} = CONVERSION RATE OF ORGANIC PHOSPHORUS
K_g = GROWTH COEFFICIENT FOR PHYTOPLANKTON
K_{S_a} = SETTLING RATE FOR PHYTOPLANKTON
K_{S_n} = SETTLING RATE FOR ORGANIC NITROGEN
K_{S_p} = SETTLING RATE FOR ORGANIC PHOSPHORUS

Figure 1. James River Model (JMSRV) Kinetics
(from Hydroscience, 1980)

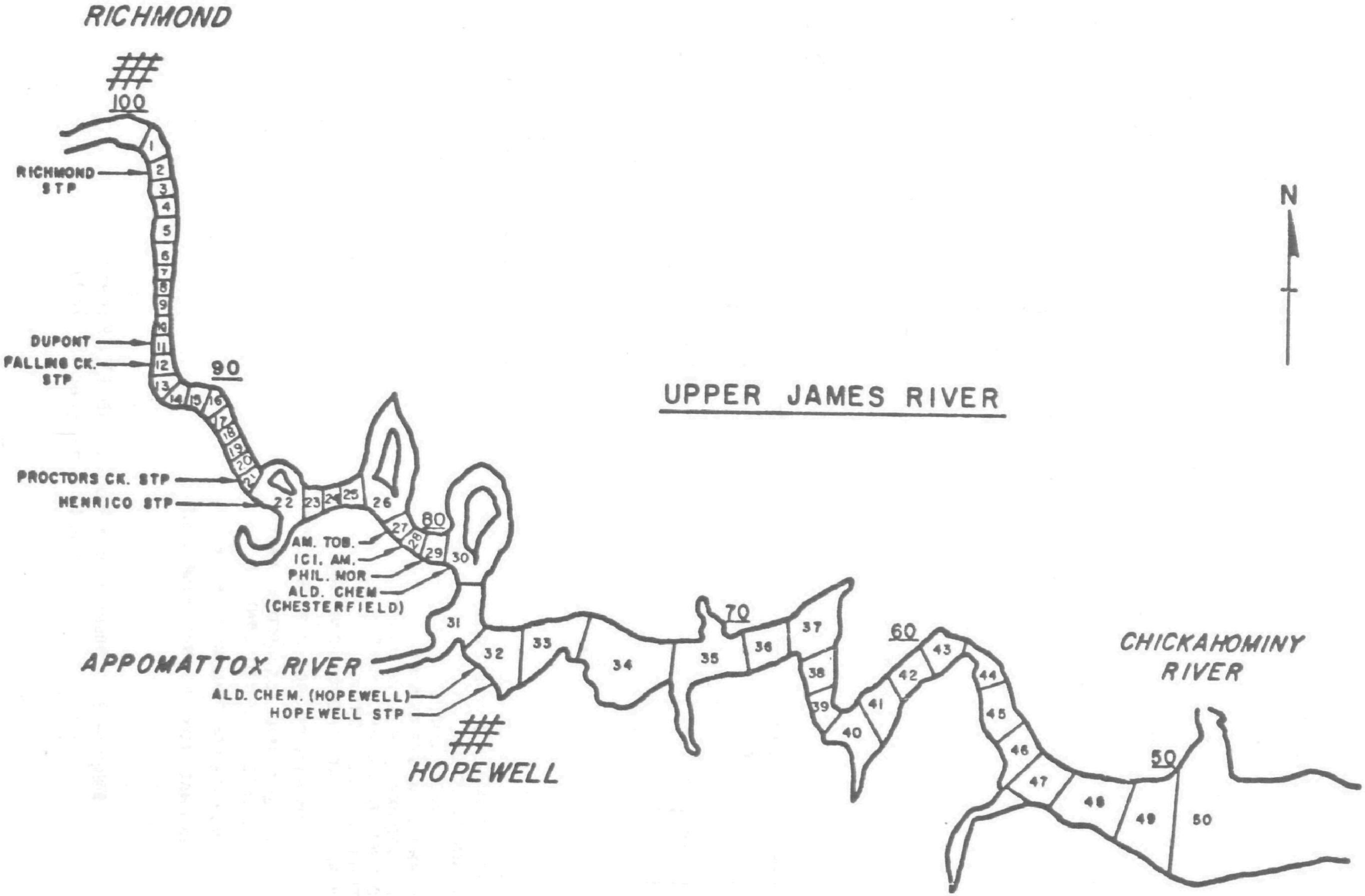


Figure 2. James River Model (JMSRV) Segmentation
(from Hydroscience, 1980)

currently available. Such a model calibration effort is necessary to update the model for changes, if any, in model coefficients (e.g., rate constants, boundary conditions, loading rates, etc.) and to understand the estuarine system under existing conditions.

The water quality data collected by the Richmond Regional Planning District Commission (RRPDC) in the summer of 1983 under the James River Water Quality Monitoring Program (2) were used in this modeling study. The data from two slack water surveys were chosen for use in this modeling study. The summer of 1983 was characterized by a prolonged period of warm temperature and low flow. The July 28 (low water slack) survey was characterized by the highest freshwater flow (combined flow near Richmond = 2380 cfs) among all surveys while the September 20 (high water slack) survey was conducted under a relatively low flow (combined flow near Richmond = 1140 cfs). The receiving water quality data and the associated point source monitoring records were utilized in this study.

The JMSRV model was used to analyze these data related to these two surveys. In addition, model sensitivity analyses were conducted to fine tune the model and to identify the factor(s) limiting phytoplankton growth in the upper James River Estuary.

As indicated earlier, the modeling analysis was conducted using a microcomputer system. A brief description of the system is included in the Appendix.

MODEL CALIBRATION AND SENSITIVITY ANALYSIS

The JMSRV model was incorporated with the hydrologic and environmental conditions of the James River associated with the September 20 survey. The point source loads ($CBOD_{40}$, organic nitrogen, NH_3, NO_2+NO_3, organic phosphorus, and ortho-phosphorus) shown in Table 1 were also incorporated into the model.

The modeling analysis assumes that the estuarine system is under an intertidal steady-state condition. In reality, however, steady-state condition rarely exists, nor does a steady dry weather river flow. In fact, the James River flow fluctuated widely on a day-to-day basis. To better approximate a steady water quality condition observed on September 20, it is necessary to use an average river flow over a period of 7 to 10 days prior to the survey. An average flow of 1100 cfs was used to best represent the flow near Richmond.

The results of model calibration using the September data are summarized in Figure 3 for $CBOD_{40}$, NH_3, NO_2+NO_3, ortho-P, chlorophyll a, and dissolved oxygen. The calculated ultimate CBOD ($CBOD_u$) concentration are compared with the observed 40-day CBOD ($CBOD_{40}$) data. The long term BOD analysis indicates complete decay of the organic materials in the river samples by 40 days. Thus, the measured $CBOD_{40}$ values closely represent the ultimate CBOD and can be compared with the calculated $CBOD_u$ without serious errors.

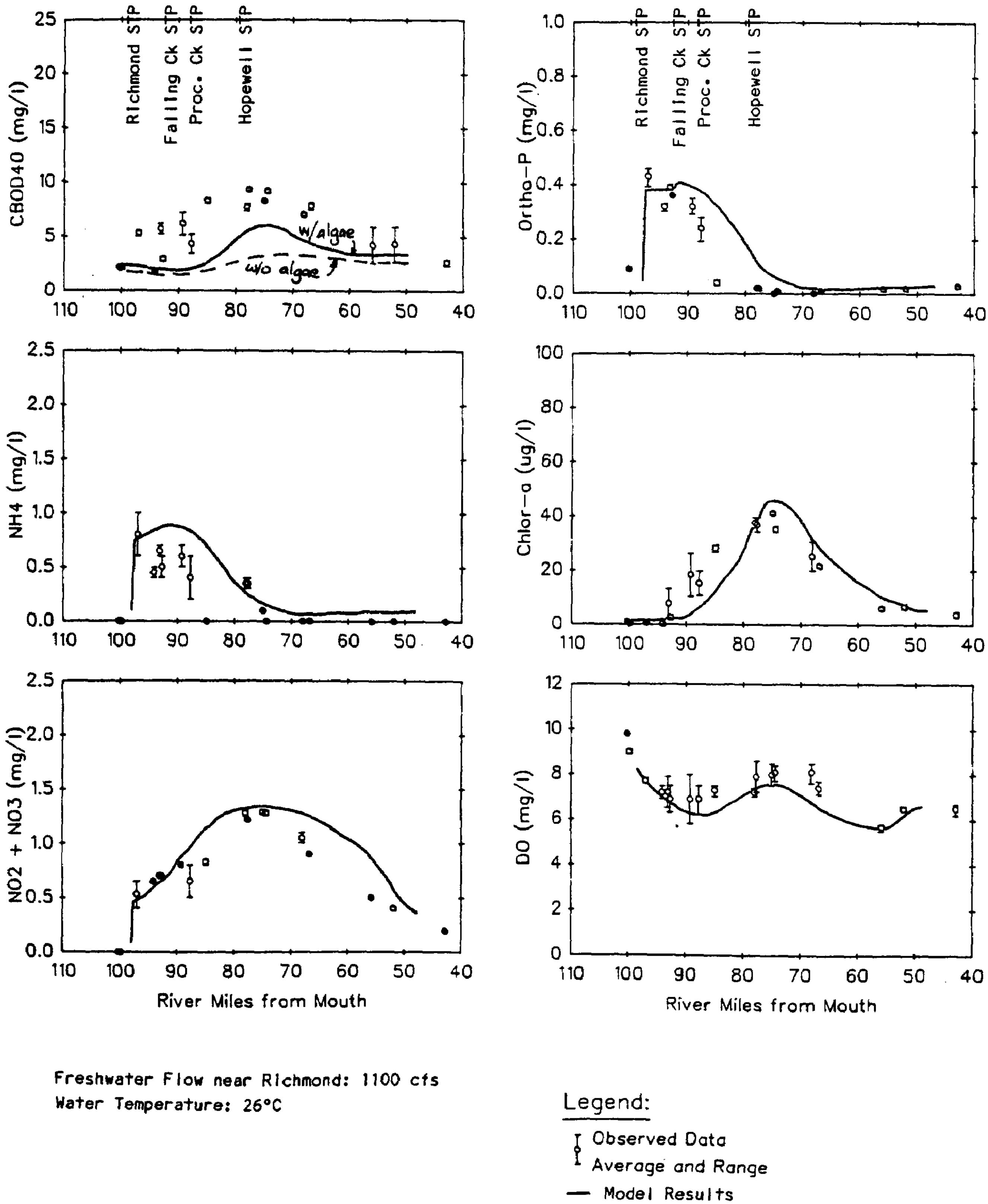


Figure 3. Model Calibration of September 20 Survey

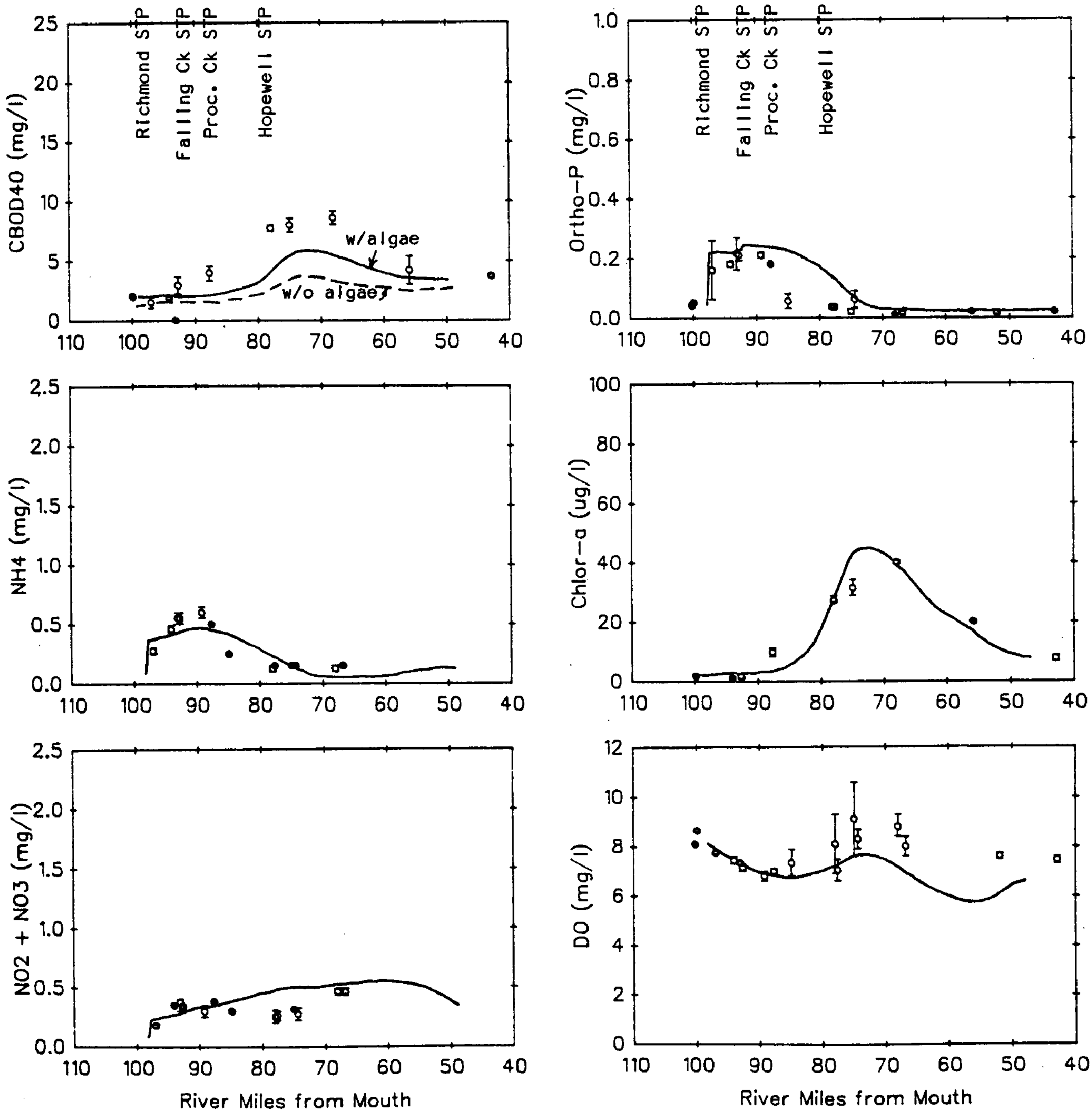
Table 1. Major Wastewater Loadings (lbs/day) for September 20, 1983 Survey

Discharger	$CBOD_{40}$	Org. N	NH_3	NO_2+NO_3	Total P	Org. P	Inorg. P
Richmond	4512.3	4927.5	3916.7	2332.9	2328.4	144.4	2184.0
DuPont	202.8	230.9	38.9	9.6	5.6	2.8	2.8
Falling Creek	714.7	336.0	116.1	745.2	502.1	111.2	390.9
Proctors Creek	2602.0	208.5	103.4	33.8	179.3	25.2	154.2
Reynolds Metals	1.8	3.9	0.0	2.1	2.3	2.2	0.2
Am. Tobacco	60.8	27.2	1.0	31.2	6.8	0.7	6.2
ICI	31.9	8.0	0.7	4.8	1.4	1.4	0.0
Philip Morris	368.4	27.0	6.7	267.7	106.2	39.4	66.8
Allied-Chester	2480.3	42.9	3.1	61.2	9.2	6.1	3.1
Allied-Hopewell	12680.9	3363.8	2069.0	2349.3	80.1	66.7	13.4
Hopewell	8929.1	7048.3	5904.6	326.8	347.7	205.2	142.5

Figure 3 indicates that the model results reproduce the observed trends of the water quality parameters. The addition of CBOD recycled from phytoplankton biomass (the CBOD curve labeled as 'with algal') matches the observed data reasonably well. Note that the CBOD curve without algal is consistently below the observed data. Thus, it is important to include the oxygen demand of decayed phytoplankton biomass in the area where phytoplankton growth is significant.

The JMSRV model was also applied to analyze the July 28, 1983 data set. Specific hydrologic (combined flow near Richmond = 2,200 cfs averaged over a week prior to the survey) and environmental conditions were incorporated along with the point source loads (Table 2) into the model. The results of model calibration are summarized in Figure 4. The model calibrations match the observed data reasonably well. Note that the relatively higher flow in July slightly reduces the nutrient concentrations while compared with the September concentrations. However, the chlorophyll a levels remain about the same between the two surveys. The major difference between the calibration results of the two data sets is in the saturated growth rate of phytoplankton (2.2/day for the September condition and 2.0/day for the July condition).

Model sensitivity analyses were designed to vary the model coefficients to reproduce the data and, therefore, would enable us to better understand the phytoplankton growth mechanisms. The final product of the sensitivity analysis is a fine tune model which would reproduce the two data sets using a consistent set of model coefficients. That is, the results from the sensitivity analysis would reaffirm the earlier model calibration results. The September survey data were used in the sensitivity analysis.



Freshwater Flow near Richmond: 2200 cfs
Water Temperature: 28°C

Legend:
Observed Data
Average and Range
Model Results

Figure 4. Preliminary Model Calibration Results (7-28-83)

The model assumed no nitrification in the James River from Richmond to Hopewell (the first 30 model segments) according to Hydroscience (1). There are, however, widespread speculations on whether nitrification is occurring in this section of the James. Nitrifying bacteria data collected in 1983 (1) could not provide a firm answer. Additional field studies to quantify the growth potential of nitrifiers are underway but their results are not available at the present time. In this modeling, a number of nitrification rates ranging from 0.05/day to 0.15/day were incorporated into the model for segments 1-30 (from Richmond to Hopewell). The results of the sensitivity analysis are presented in Figure 5. The nitrification rate of 0.05/day (at 20°C) gives the best fit to the data among the rates tested, considering the reproduction of the NH_3, NO_2+NO_3, and DO data.

Table 2. Major Wastewater Loadings (lbs/day) for July 28, 1983 Survey

Discharger	$CBOD_{40}$	Org. N	NH_3	NO_2+NO_3	Total P	Org. P	Inorg. P
Richmond	5642.1	1282.7	3216.5	1379.2	2314.7	144.7	2170.0
DuPont	427.5	217.3	0.0	63.1	12.6	6.3	6.3
Falling Creek	1067.2	398.0	328.8	311.5	461.5	109.6	351.9
Proctors Creek	312.5	312.5	45.3	36.2	156.2	64.3	91.9
Reynolds Metals	3.3	0.7	0.0	0.9	0.6	0.4	0.2
Am. Tobacco	16.3	60.1	14.3	3.1	40.3	17.6	22.7
ICI	17.9	8.0	0.6	4.6	1.4	1.4	0.0
Philip Morris	485.7	26.7	8.8	351.4	140.0	52.0	88.0
Allied-Chester	3859.1	46.5	3.6	35.7	0.0	0.0	0.0
Allied-Hopewell	16502.0	1163.3	1055.1	1514.9	60.9	47.3	13.5
Hopewell	10347.6	5046.9	6989.5	429.5	322.1	119.1	203.0

Based on the preceding nitrification analysis, the model was then tested with different growth rates of phytoplankton ranging from 2.0/day to 3.0/day. The results of the sensitivity runs are shown in Figure 6. At the lowest algal growth rate of 2.0/day, nitrogen is shifted from the phytoplankton biomass to NO_2+NO_3, resulting in lower chlorophyll _a_ level and slightly lower dissolved oxygen concentrations in the area between river miles 70 and 90. On the other hand, a growth rate of 3.0/day produces a chlorophyll _a_ peak about 85 ug/l. A growth rate of 2.2/day seems to produce a close fit of the September data (Figure 6).

Hydroscience (1) suggested that the losses of ammonia nitrogen and ortho-phosphorus between rives miles 90 and 80 may be due to inorganic nutrient uptake by rooted aquatic weeds in the marshes and oxbows in this stretch of the river. Since no data is available to confirm this hypothesis, an empirical approach is taken in this analysis to incorporate a loss rate of

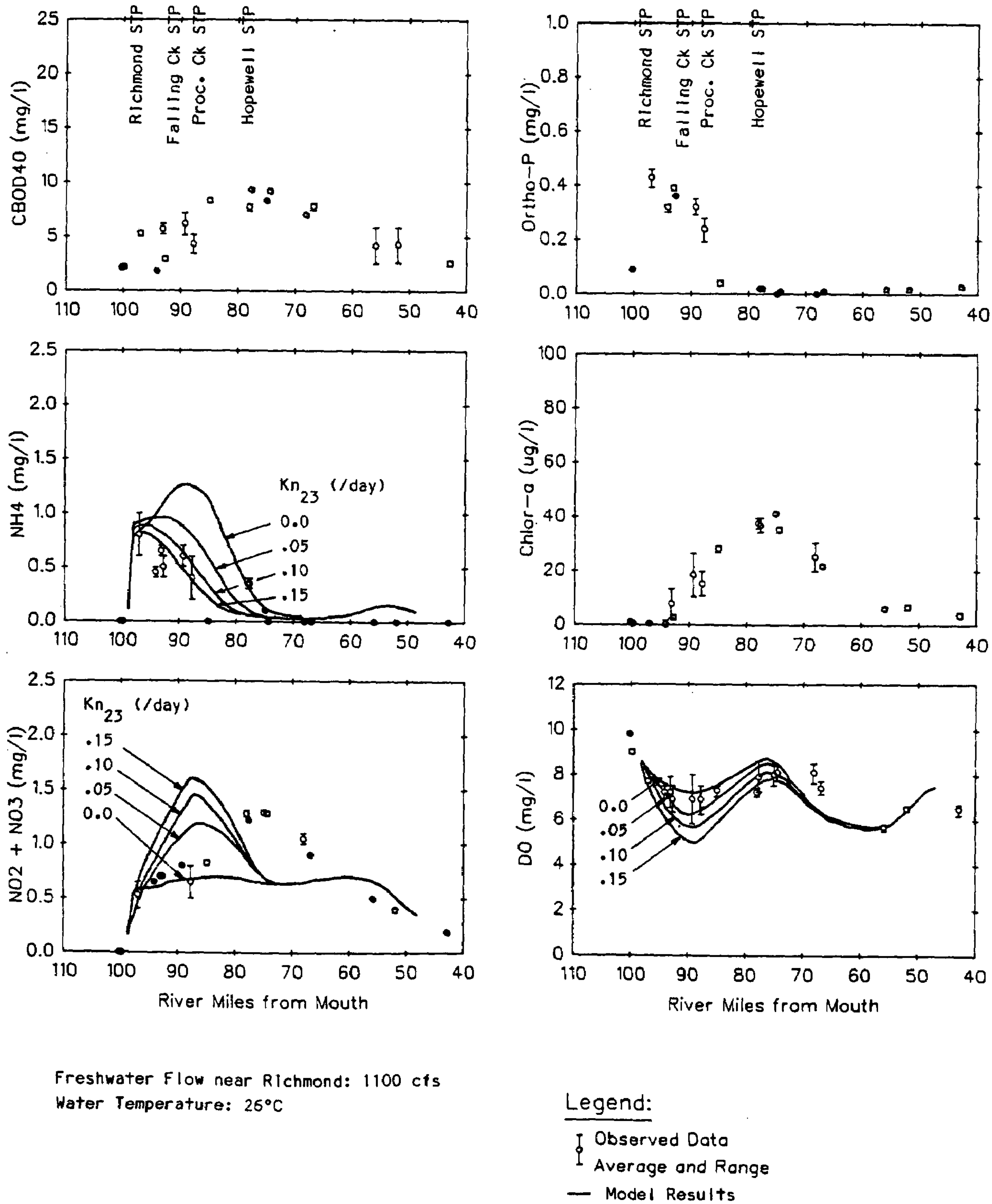


Figure 5. Model Sensitivity Results - Nitrification

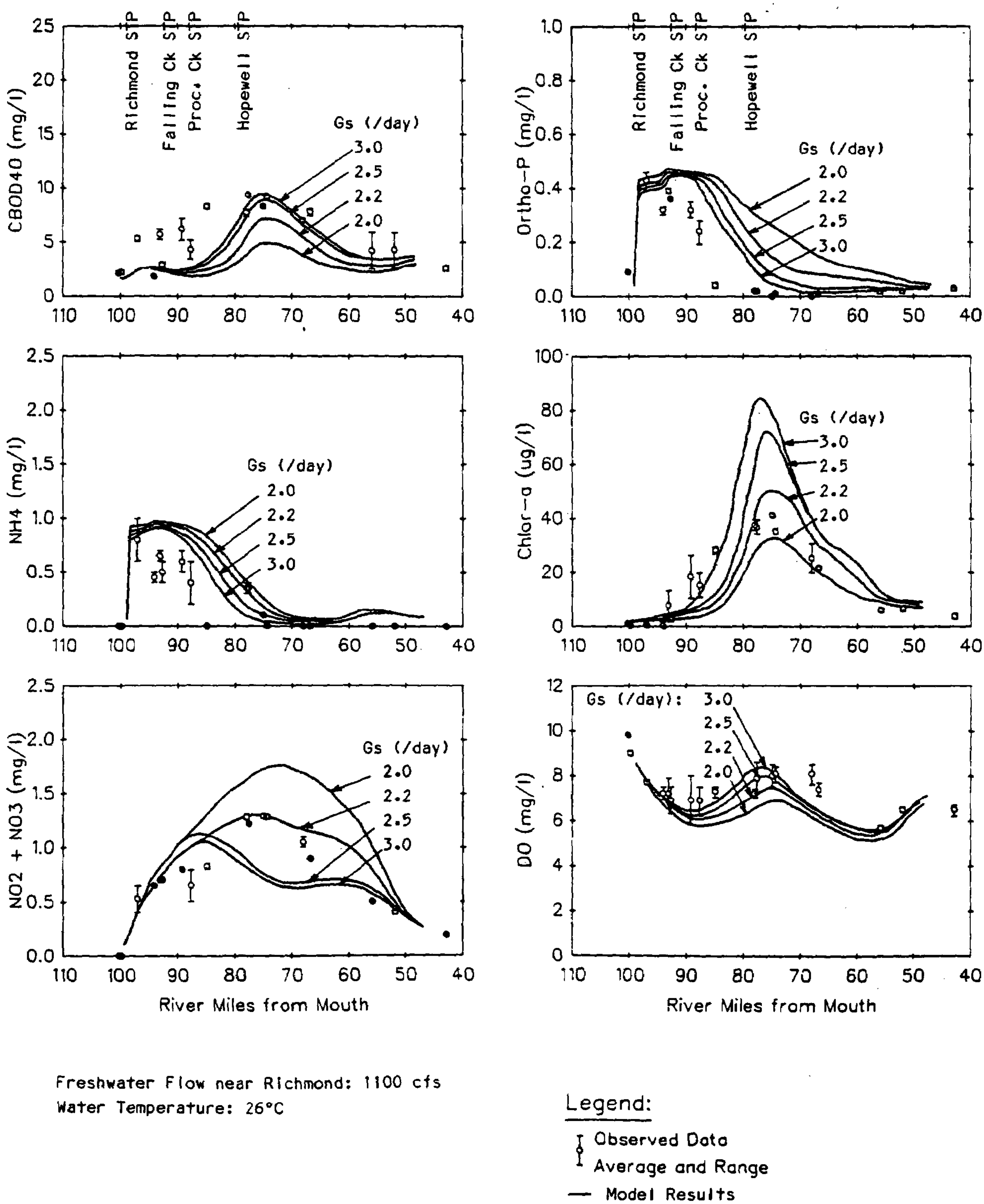


Figure 6. Model Sensitivity Results - Phytoplankton Growth Rate

ortho-phosphorus (0.5 ft/day) into the model. Such a loss rate may include not only the uptake by aquatic weeds, but also some other mechanisms such as phosphorus adsorption by sediments. Figure 7 shows that incorporating such a loss rate brings the calculated ortho-phosphorus concentrations closer to the data.

DISCUSSIONS

The JMSRV model is now calibrated reasonably well (see Figures 3 and 4) with a consistent set of model coefficients (see Table 3) using two data sets from 1983. Additional insights into the estuarine system can be summarized from the model calibration and sensitivity analysis results. First, the location of the phytoplankton biomass peak moves up and down the estuary with the freshwater flow. Between the two water quality surveys, the July condition (associated with a freshwater of 2,200 cfs in Richmond) produced a phytoplankton biomass peak near river mile 70. On the other hand, the lower freshwater flow of 1,100 cfs in the September survey moved the peal upestuary to river mile 75.

The question of nutrient limitation can be explored from the model results. Figure 8 shows the degrees of nutrient limitation (nitrogen and phosphorus) on phytoplankton growth in July and September, 1983. Both nitrogen and phosphorus are not limiting the growth rate as the Michaelis-Menton limitation ratios are close to 1.0 (practically no reduction in growth rate). Further, light appears to be a major factor in reducing the growth rate. Figure 9 shows that the specific growth rates of phytoplankton (/day) are significantly reduced in the turbid water along the estuary in September, 1983. In an earlier study on the lower James River Estuary, Neilson and Ferry (4) suggested that factors (other than nutrients), such as turbidity, mixing, and zooplankton grazing, are likely to control phytoplankton growth.

SUMMARY AND CONCLUSIONS

The JMSRV model has been calibrated using the most recent data currently available on the upper James River Estuary. The model sensitivity analyses indicate that the calculated phytoplankton peak in the upper James River Estuary is sensitive to the saturated growth rate of phytoplankton. The model calibration results also indicate that nutrients (nitrogen and phosphorus) are the key limiting factor for phytoplankton growth. Rather, light or turbidity is the major limiting factor under existing conditions. A similar finding was stated by Neilson and Ferry (4) in an earlier study of the lower James River Estuary.

The modeling analysis was conducted on a microcomputer system. The execution of the compiled program proves to be very efficient with the help of a math co-processor 8087. In addition, specially designed software programs for the microcomputer system were used to generate the output in graphical form using a HP plotter. The entire operation using the microcomputer system offers many advantages over the execution on some mainframe systems.

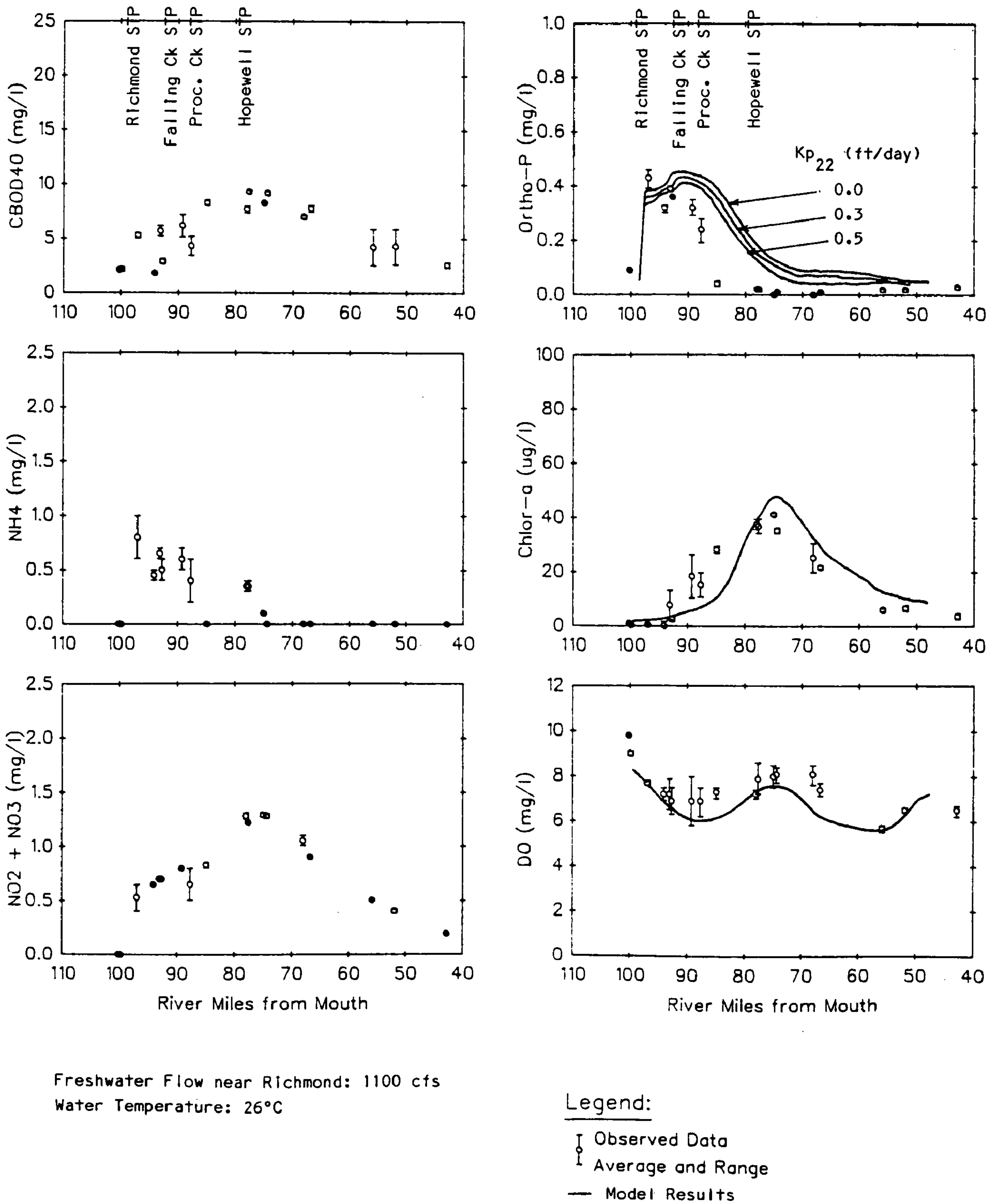
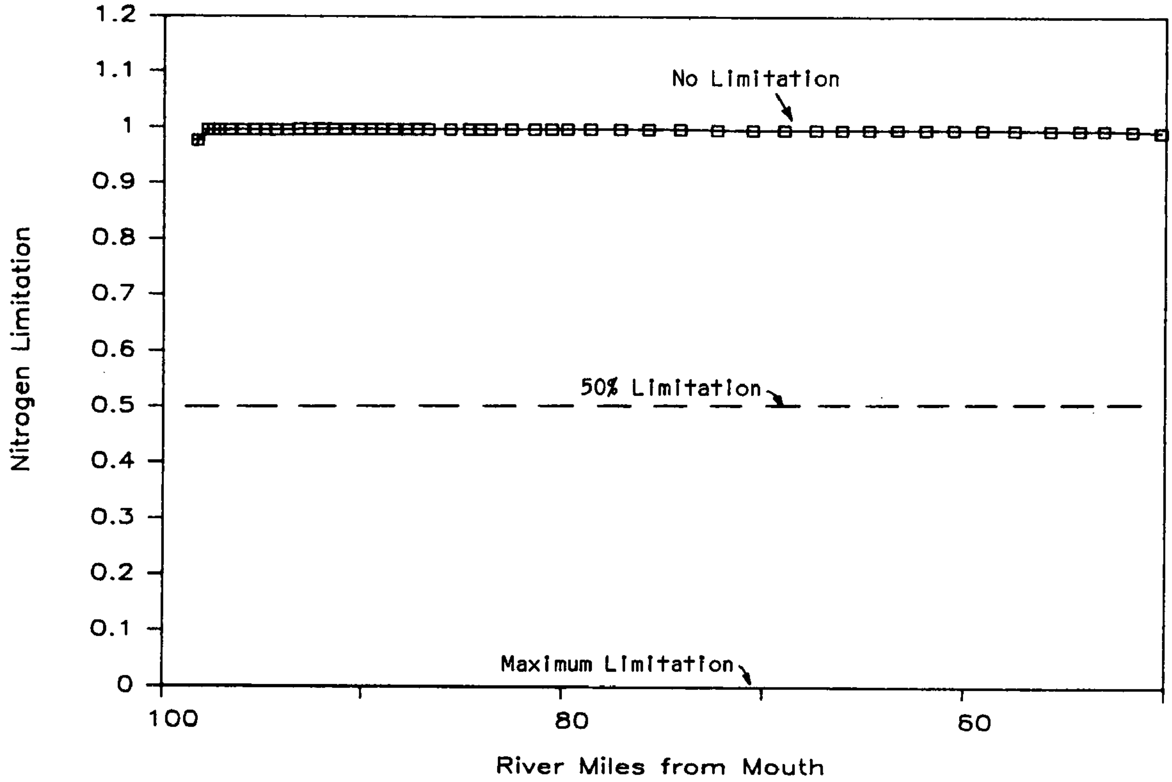


Figure 7. Model Sensitivity Results - Nutrient Uptake by Weeds



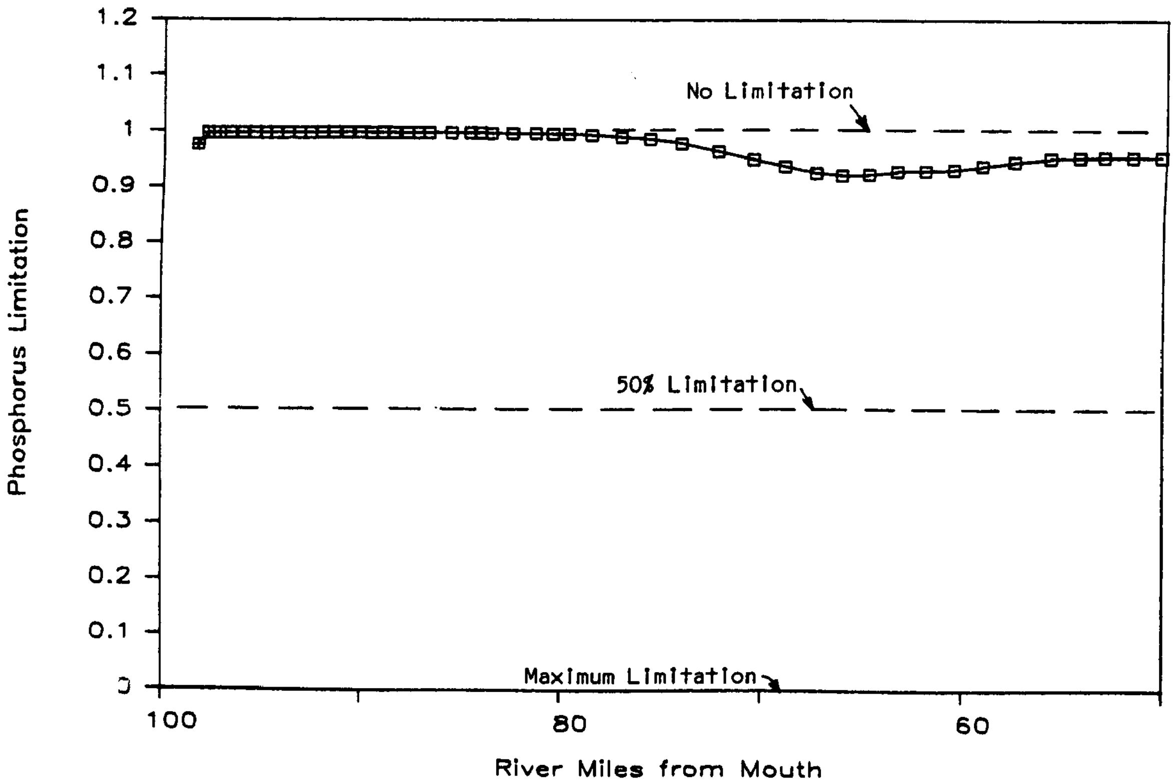


Figure 8. Degree of Nutrient Limitation

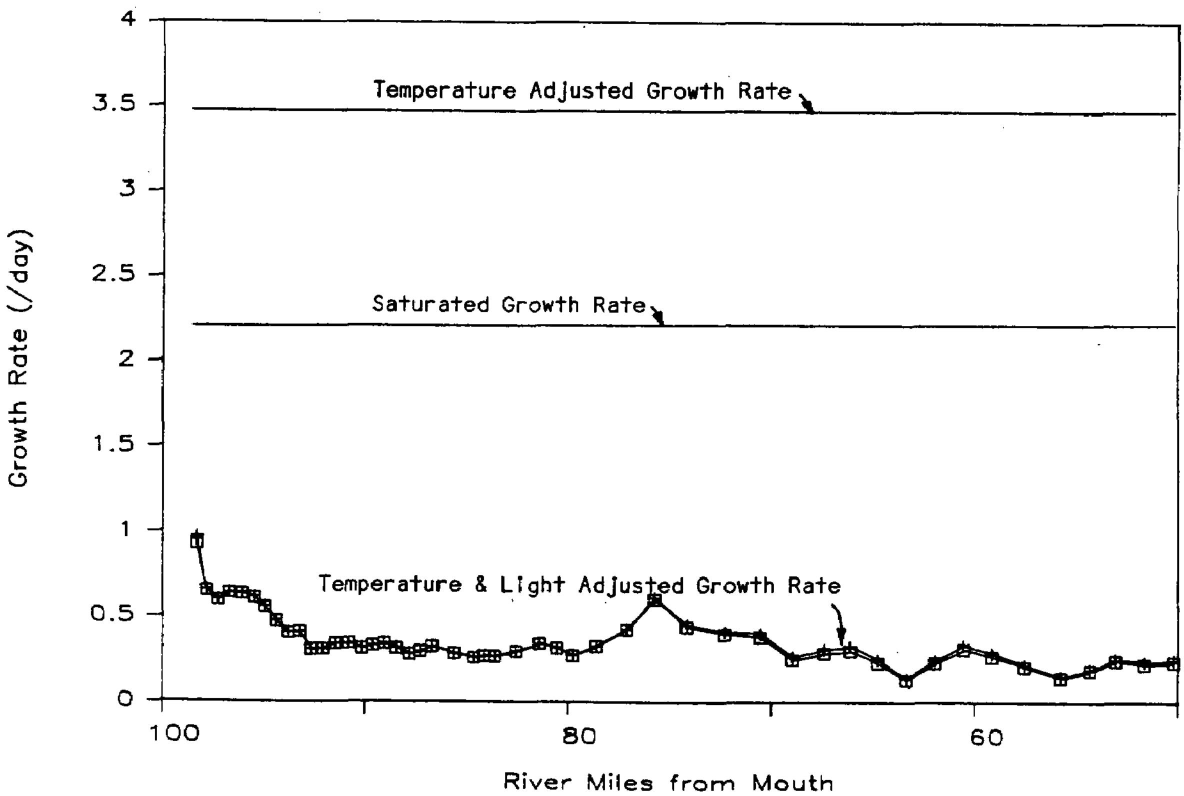


Figure 9. Specific Growth Rate of Phytoplankton

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Table 3. James River Model Parameters

Kinetics Coefficients (Base e @ 20°C)

Oxygen Transfer	ft/day	3.00
Deoxygenation	1/day	0.10
Nitrification	1/day	0.05 (Segments 1-30) September Survey
		0.00 (Segments 1-30) July Survey
		0.15 (Segments 31-50)
Hydrolysis - N	1/day	{0.10 (Segments 1-30)
		0.15 (Segments 31-50)
- P	1/day	{0.05 (Segments 1-30)
		0.10 (Segments 31-50)
Setting - N	ft/day	0.75
- P	ft/day	0.75
- Chl 'a'	ft/day	0.75
Growth	1/day	2.20
Respiration	1/day	0.10
Death	1/day	0.10
Extinc. Coef.	1/meter	1.4 (Segments 1-10)
		2.0 (Segments 11-31)
		2.3 (Segment 32)
		3.0 (Segments 5-6)
Hrs. of Daylight	hrs	12.0 (September)
		14.0 (July)
Benthic Demand	gm/m^2-day	0.5 (Segments 1-30)
		1.5 (Segments 31-50)

Stoichiometry & Constants

Temperature	°C	26.0 (September)
		28.0 (July)
C/CHL Ratio	mg/µg	0.025
N/CHL Ratio	mg/µg	0.007
P/CHL Ratio	mg/µg	0.001
O_2/C Ratio	mg/mg	2.67
Half. Sat. Conc. - N	mg/l	0.005
- P	mg/l	0.001
Sat. Light	langleys/day	300.
Avail. Light	langleys/day	600. (September)
		450. (July)

APPENDIX - MICROCOMPUTER USED FOR THIS STUDY

HARDWARE

The system hardware consists of a COMPAQ (IBM compatible) computer with the following configuration:

- 256K RAM
- 16-bit Intel 8088
- Math Coprocessor 8087
- Dual 320K disk drives
- Serial port and parallel port
- Graphics

The computer comes with a 9" monochrome monitor. In addition, a GEMINI 10X dot matrix printer is connected with the COMPAQ via the parallel port. A HP 7470A plotter is connected with the computer through the serial port.

SOFTWARE

The following software programs were used in this study:

- JMSRV water quality model
- FORTRAN 77 compiler which supports 8087 math coprocessor
- LOTUS
- Specially developed plotting programs used with the HP 7470A plotter to generate data and model result plots

PERFORMANCE

The source code of the JMSRV model (size about 70K) was compiled using the Microsoft FORTRAN 77 compiler which has a nice feature of supporting the 8087 math co-processor. The math co-processor, designed to handle float point executions, is very important in achieving reasonable program execution speed. In this case, the execution of the compiled program takes about 4 hours to complete a run (i.e., 120-day time variable run at a 0.5-day time step) on the COMPAQ prior to the installation of the 8087 math co-processor. Once the 8087 is in place, it takes 11 minutes to complete the same model run. The reduction of run time in this case makes the use of microcomputers for this study and many others a very attractive alternative to using the mainframe.

Of course, a typical run like that one usually takes no time on a main-frame computer. However, it is the other software features which offer a user friendly environment, making the use of a micro even more appealing. For example, the LOTUS program can be used to view any model results in the form of plots and can generate some simple plots using the HP plotter. The final plots shown in this paper were generated by a specially written program for the HP plotter. Thus, a number of key water quality constituents can be presented on a single plot to tell the whole story.

APPLICATION OF FACTOR ANALYSIS TO MANAGEMENT OF IMPOUNDMENT WATER QUALITY

by: Curtis D. Pollman
Robert E. Dickinson
Environmental Science and Engineering, Inc.
Gainesville, Florida 32602

ABSTRACT

An inherent problem in monitoring studies of water quality is reducing the dimensionality of the data so that a determination of processes or factors controlling water quality can be made. One approach is principal components analysis (PCA) which constructs from the data a few important factors from all the measured variables by taking advantage of intra-variable correlations. PCA was used to derive water quality factors from over 40 chemical parameters measured in a series of reclaimed phosphate pit lakes located in central Florida. The relationship between the extracted factors and reclaimed lake morphometry and hydrology were evaluated through multiple linear regression.

Extracted factors generally fall within a well-defined typology, i.e. lake trophic state factor, inorganic sediment factor, inorganic nitrogen factor, etc. The strongest correlation with the modeled morphometric and hydrologic variables was observed for the factor reflecting the major dissolved constituents in the water column, followed by the trophic state factor. Multiple regression analysis suggests that water quality in reclaimed lakes is largely controlled by in-lake processes. For example, the lake trophic factor was negatively correlated with maximum lake depth, indicating that sedimentation of detritus as a nutrient sink is important in regulating trophic state. In fact, overall data variability appeared most strongly influenced by the two depth variables, maximum and average depth, followed by lake volume and volume development. Lake hydrology was somewhat less important as was lake orientation to prevailing winds.

INTRODUCTION

In October 1981, the Florida Institute of Phosphate Research (FIPR) sponsored an extensive research program to study 12 artificial lakes created during the course of reclamation of mined phosphate pits in central Florida. Data on the physical, chemical, and biological conditions of each lake were collected during a 1-year sampling program.

One aspect of the data analysis program was to examine the influence of lake morphometric and hydrologic features on lake water quality. An analytic procedure, principal components analysis (PCA), was used to create "factors" that would reduce the dimensionality of the data. These factors were then correlated with morphometric and hydrologic parameters to find influential parameters.

The purpose of the PCA ultimately was the creation of lake design criteria. Given controllable features such as mean depth, maximum depth, and shoreline configuration, how should reclaimed lakes be designed to maximize lake water quality? This analysis in turn can be applied to any manmade lake such as stormwater detention ponds and water supply reservoirs.

PRINCIPAL COMPONENTS ANALYSIS

The FIPR reclaimed lake study had 80 measured water quality and biological variables. The large number of variables would render the determination of how lake morphometric and hydrologic characteristics affect lake water quality and biology a tedious task without the aid of a simplifying analytic procedure. A PCA constructs a few important "factors" from the many variables by using the intra-correlations of the variables to reduce the dimensionality of the problem. The resulting factors, each a linear combination of all the variables, are uncorrelated with one another, and a few factors often account for most of the variation of the entire set of variables.

The meaning of a particular factor is determined by considering those variables that "load" highly on the factor. The variable loadings may be interpreted as correlation coefficients. They range from -1 to 1, with -1 indicating perfect negative correlation, 1 a perfect positive correlation, and 0 no correlation.

The variables were first transformed by using the ln transform and controlling both seasonal (sampling trip) and spatial (station) variability before using the principal component analysis. Seasonal and spatial variability were controlled by modeling each variable as a function of season and station and calculating the least-squares residual. The least-squares residuals are the differences between the actual observation and the mean for the corresponding season-station combination.

The variables were divided into chemical and biological subsets, and separate PCA's were performed for the set of chemical or abiotic variables and the set of biological or biotic variables. This separation of analysis groups aided in the interpretation of the factors. Only the results of the abiotic PCA will be discussed in this paper.

The PCA was then performed on the transformed residuals by employing an iterative process. The extracted factors were evaluated at each iteration, and all variables which did not contribute strongly to any factor were assumed to be relatively minor components and removed from further iterations. This process was continued until only contributory variables were retained.

The factors from the PCA were subsequently subjected to stepwise multiple linear regression with a set of morphometric/hydrologic parameters. The selected parameters were features that can be reasonably controlled during the course of reclamation design and encompassed all aspects of lake morphometry and hydrology. The parameter list consisted of:

- o AREA = total lake surface area (ha)
- o FLTOT = hydraulic loading rate (m/yr)
- o TIME = hydraulic residence time (yr)
- o SDI = shoreline development index
- o VDI = volume development index
- o VOL = total lake volume (m^3)
- o WSIA = annual wind stress index (orientation parameter)
- o Z = mean lake depth (m)
- o ZMAX = maximum lake depth (m)

The stepwise multiple regression approach was used as a model-building process to determine which combinations of independent parameters were highly correlated with water quality factors. The procedure used each of the water quality factors (determined by PCA) as a dependent variable and then added independent parameters to the model in their order of importance as explanatory parameters. Parameters were added as long as their contribution to the model was statistically significant at the alpha = 0.15 level. The stepwise model building procedure acted as a filter to screen the most important independent parameters. It was not intended to provide a predictive model but to indicate strong associations.

The results of PCA on total analytical concentrations of the various chemical parameters are presented in Table 1, which is restricted to those loadings exceeding or equal to 0.50. Individual eigenvalues and the percent of the total variance explained by each factor are also included in the table. Over 50 percent of the total variance in the data set was explained by the first four factors, with Factors 1 and 2 accounting for 33.3 percent. Factor 1 is related to the inorganic constituency of sediments in the reclaimed lakes, with sediment-associated chromium (SCr) and total phosphorus (Total P) loading most highly on the factor (r = 0.93 and 0.87). Sediment moisture content, which is indicative of the energy state or depositional environment prevailing at the sediment-water interface (1,2, and 3), also loaded relatively highly on Factor 1, although the correlation was weaker (r = 0.56).

The second factor reflects primarily major dissolved components in the water column. Total dissolved solids (TDS) and conductivity are indicative of the ionic strength of the water column and are strongly correlated with Factor 2 (r = 0.88 and 0.74, respectively). Calcium (Ca) and magnesium (Mg) dominate the factor with alkalinity and, to a lesser extent, fluoride (F) associated with Factor 2 as the counter ions. Sulfate (SO_4), which is the

TABLE 1. EIGENVALUES, PERCENT VARIANCE EXPLAINED, AND LOADINGS FOR PCA FOR RECLAIMED LAKE CHEMISTRY PARAMETER

Variable	Factors									
	1	2	3	4	5	6	7	8	9	10
Moisture	0.59			0.56						
SCa*	0.61									
SMg	0.56									
SK	0.82									
SBa	0.84									
STP	0.87									
SCr	0.93									
SPb	0.82									
SCd	0.77									
SeRa226	0.82									
SSr90	0.75									
pH		0.59								
Conductivity		0.74								
TDS		0.88								
Ca		0.94								
Mg		0.97								
Alkalinity		0.76								
F		0.64								
TOC		0.58								
Turbidity			0.67							
Secchi			-0.84							
Total P			0.65							
Total N			0.61							
SO_4			-0.63							
STOC				0.86						
STON				0.77						
SNH_3				0.57						
SCl				0.62						
Cd					0.78					
SHg					-0.73					
SOP					0.78					
K						0.68				-0.54
Ra 226						-0.71				
SSO_4						0.76				
SNO_x						0.69				

TABLE 1. EIGENVALUES, PERCENT VARIANCE EXPLAINED, AND LOADINGS FOR PCA FOR RECLAIMED LAKE CHEMISTRY PARAMETER (CONTINUED, PAGE 2 OF 2)

Variable	Factors 1	2	3	4	5	6	7	8	9	10
SAs							0.88			
SPb							0.76			
NH_3								0.72		
NO_x								0.85		
SSe									0.88	
DO_2										0.76
Eigenvalue	7.44	6.55	4.19	3.50	3.24	2.88	2.50	1.82	1.50	1.33
Percent Variance	17.7	15.6	10.0	8.3	7.7	6.9	6.0	4.3	3.6	3.2
Cumulative Variance	17.7	33.3	43.3	51.6	59.3	66.2	72.1	76.5	80.0	83.2

*Variables preceded by an S are for sediment data (i.e., SCa = sediment calcium).

predominant anion on an average basis for the 12 reclaimed lakes, loaded rather weakly on Factor 2 despite its close correlation with Ca (r = 0.54).

The third factor extracted from the principal component analysis is indicative of trophic state and is similar to trophic state factors derived by Shannon (4) and Preston (5) for other Florida lakes. The nutrient forms, total nitrogen (Total N) and Total P, as well as parameters indicative of light transparency (i.e., turbidity and secchi disk transparency) are associated with this factor.

The strongest loading on Factor 3 is Secchi disk transparency, which loads negatively on the factor and implies reduced-light transparencies in enriched or eutrophied reclaimed lakes. More difficult to explain is the association of SO_4 with this factor. A priori considerations suggest that variability in SO_4 levels would be most closely associated with a factor related to ionic distribution and weathering (i.e., Factor 2). The correlation of SO_4 with Factor 3 may reflect SO_4 reduction and depletion in hypolimnetic waters, a process (6) recently demonstrated to be rather significant in the hypolimnion of a productive, softwater lake.

The fourth factor, which accounts for 8.3 percent of the total variance in the data, is essentially an organic matter deposition factor and relates those variables that tend to be most influenced by the accumulation of sedimentary organic matter. The highest loading is for sedimentary total organic carbon (STOC), followed by sedimentary total organic nitrogen (STON). Also associated with Factor 4 is interstitial ammonia (SNH_3) which builds up in the pore water as a direct consequence of catabolic processes within the sediments.

It is interesting to observe that sediment moisture content loads in a positive sense for Factor 4 as well as Factor 1, which relates inorganic sediment variables. Previous studies have demonstrated a close relationship between sedimentary water content and organic content (7, 8, and 3); however, sediment moisture content also reflects selective size sorting processes and increases with decreasing particle size due to interparticle repulsion (9). This implies that the sedimentary accumulation of trace elements identified in Factor 1 is the result of adsorption to clay particles, which generally carry a net negative charge and have larger surface area:mass ratios than coarser particles.

Of the remaining factors that exceed the minimum eigenvalue threshold level of 1.0, Factors 8 and 10 are of the most interest. Factor 8 is essentially an inorganic nitrogen factor and comprises ammonia (NH_3) plus nitrate- and nitrite-nitrogen (NO_x) forms. Factor 10 is mainly the result of the positive correlation of dissolved oxygen and the negative correlation of dissolved radium-226 (Ra-226). Factors 7 and 9 result from the correlation of sedimentary arsenic (SAs) and lead (SPb) (Factor 7) and sedimentary selenium (SSe) (Factor 9). The remaining factors (Factors 5 and 6) are complex and difficult to interpret with respect to underlying causes.

ABIOTIC FACTORS AND LAKE MORPHOMETRY

The relationship between morphometric features and reclaimed lake chemistry was investigated using stepwise multiple linear regression; the abiotic factors were the dependent variables, and the lake morphometric and hydrologic parameters were the independent variables (see Table 2 for results).

From a trophic state perspective, the most interesting model is represented by the relationship between Factor 3 and the lake morphometry/hydrology. The parameters FLTOT, VDI, Z, and ZMAX were all found to be significantly correlated ($P < 0.0001$) with the trophic state factor (Factor 3). Inclusion of other variables in the regression equation did not sufficiently improve the coefficient of determination ($r^2 = 0.50$) to justify increasing the complexity of the model. Inspection of calculated F values for each variable indicates that lake maximum depth and volume development index exert the greatest influence on the factor. Of these two, the most important variable in the relationship is ZMAX, which was negatively related to the factor. The relationship between the trophic state factor and ZMAX agrees with established limnological principles, viz., all other factors being equal, overall lake nutrient levels should decrease with increasing depth because of the reduced rate of nutrient recycling across the sediment water interface. Furthermore, sediment focusing may occur in lakes with deep holes, effectively removing detrital material from interacting with the trophogenic zone of the water column.

The positive correlation of mean depth with the trophic state factor conflicts with the influence of maximum depth and further illustrates the relative importance of deep holes on reclaimed lake chemical dynamics. Within a particular class of lakes, internal loading of nutrients in response to disturbances at the sediment-water interface created by wind-driven circulation and other processes tends to diminish in importance as mean depth increases. This, of course, reflects the greater amount of energy that must be applied at the lake surface for wind-induced wave energy to extend to the bottom in deeper lakes. The net effect of the sediments as an ultimate sink for nutients, therefore, increases with mean depth (12, 13).

The fact that reclaimed lakes with greater mean depths tend to have increasing values for the trophic state factor is indicative of the extremely shallow nature of these lakes and suggests that other factors beyond lake hydrodynamics and simple increases in assimilative capacity are associated with changes in mean depth. Mean depth for the 12 lakes ranged from 1.8 to only 5.4 meters (m). Chapra (14) indicates that the effects of sediment resuspension is probably significant in lakes with a mean depth of 10 m or less. It seems likely, therefore, that sediments throughout the basins for each reclaimed lake are periodically disturbed and resuspended, with deposition and removal occurring only in deep holes. The resultant effect of deep holes would be increased water clarity because of lower turbidity levels and decreased rates of algal productivity.

Within the depth range of the reclaimed lakes surveyed, it is apparent that increasing $\bar{z}$ has virtually no distinguishable effect on internal loading processes. The positive effect of mean depth on the trophic state factor instead suggests that penetration of the lake basin into the phosphatic bedrock underlying the surficial unconsolidated layer of sands contributes to the trophic state of reclaimed lakes. Reclaimed lakes with shallower mean depths included in this study generally were constructed by filling the initial mine pit with a relatively larger volume of overburden; consequently, these lakes are less influenced by residual tailings and clay.

Hydraulic loading rate (FLTOT) also affects the trophic state factor in a negative sense, although the relatively low F value indicates that the role of this parameter is not as important as the physical structure of reclaimed lakes. The negative relationship between FLTOT and the trophic state factor may simply reflect the effects of dilution on nutrient inputs. In other words, considering two lakes with similar external mass loads of Total N and Total P, the lake with the larger hydraulic loading rate correspondingly will have reduced input nutrient concentrations and will be characterized by a lower trophic state as well. This relationship also may imply that internal recycling or loading is important in maintaining high rates of productivity in reclaimed lakes and that the relative importance of internal processes in controlling trophic state diminishes as the hydraulic loading rate increases.

Lake trophic state is also related to Factor 8, which reflects inorganic nitrogen concentrations. Because of the high levels of phosphate typical of reclaimed phosphate pit lakes, nitrogen availability may be hypothesized to limit primary production. A substantial portion of the dispersion in the inorganic nitrogen factor can be accounted for by the morphometric/hydrologic variables FLTOT, TIME, and Z. Multiple linear regression of the factor against these variables yields r^2 equal to 0.42 ($p<0.0001$) (Table 2).

The sign of the coefficients in the multiple regression model indicates that in-lake nitrogen concentrations increase with the hydraulic loading rate and residence time (Table 2). The relationship of the hydraulic loading rate, FLTOT, in the model strongly suggests that inorganic nitrogen levels in reclaimed lakes result primarily from allochthonous sources. In addition, the positive relationship of residence time in the model for the inorganic nitrogen factor suggests that reclaimed lake water quality is in a transitional state. This should not be construed that, ceteris paribus, lakes with long residence times will necessarily have higher inorganic nitrogen concentrations than rapidly flushed lakes. Immediately after the formation and inundation of an artificial basin, a pulsed release of nitrogen usually accompanies the decomposition of submerged vegetative matter (16). At this point, in-lake nitrogen concentrations exceed levels supported by external inputs and the lake will approach a lower, steady-state condition at a rate inversely related to the hydraulic residence time. Once reclaimed lakes mature and approach steady-state conditions, lakes with longer residence times will characteristically have lower nutrient levels because of the nonconservative behavior of nitrogen and phosphorus. Biological uptake of nitrogen and phosphorus results in a net loss of these substances from the water column to the sediments. Apparent settling velocities are on the order of 5.5 and 8.5 meters per year (m/y) for nitrogen and phosphorus, respectively (13); thus, net removal of nutrients increases with lake detention time.

Multiple regression analysis indicates that ionic content or salinity in reclaimed lakes, as represented by Factor 2 (Table 2), is largely controlled by in-lake processes. F statistics for this model, which account for 77 percent of the dispersion in the salinity factor, indicate that significance in the model is primarily attributable to mean depth and the wind stress index. The positive correlation of mean depth with the salinity factor suggests that penetration of the lake basin into the calcareous bedrock underlying the surficial unconsolidated layer of sands controls the ionic character of the water column. This is supported by inspection of the components that comprise the salinity factor. This factor is dominated by the positive associations of Mg ($r = 0.97$) and Ca ($r = 0.94$), which are derived primarily from the bedrock. Furthermore, cation balances show that the reclaimed lakes contain primarily Ca and, to a lesser extent, Mg. Sodium (Na), which is derived principally from atmospheric inputs, and potassium (K) are considerably less prevalent. It is important to note that unlike inorganic nitrogen, the ionic constituency of reclaimed lakes is not controlled to an appreciable extent by hydraulic residence time.

Somewhat more difficult to interpret is the influence of maximum depth on variability in the salinity factor. In the desired statistical model (Table 2), the effect of ZMAX is negative--opposite in sign to mean depth. This dichotomy in effect by these two parameters was also observed for the trophic state factor and is apparently indicative of the importance of deep holes as traps or sinks for detrital or easily weathered material. Thus, assuming that weathering within the lake basin dictates the ionic composition of reclaimed lakes, it follows that the wind stress index is positively

TABLE 2. MULTIPLE LINEAR REGRESSION RELATIONSHIPS BETWEEN ABIOTIC PCA FACTORS AND LAKE MORPHOMETRY/HYDROLOGY

Factor Characteristic	Factor No.	Significant Morphologic Features	R^2
Sediment Inorganic Constituents	1	= 1.26 - 0.151 (ZMAX)	0.14
Water Column Ionic Content	2	= 8.65 - 9.41 (VDI) + 1.82 (WSIA) + 3.43 (Z) - 1.25 (ZMAX)	0.77
Trophic State Indicators	3	= 9.86 - 0.0930 (FLTOT) - 7.04 (VDI) + 2.08 (Z) - 0.924 (ZMAX)	0.50
Sediment Organic Matter Deposition	4	= -1.34 + 0.814 (VDI) + 0.00274 (VOL)	0.42
Water Column Inorganic Nitrogen	8	= -1.33 + 0.291 (FLTOT) + 1.11 (TIME) - 0.359 (Z)	0.37
Dissolved Oxygen/ Radium-226	10	= -0.272 + 0.0672 (FLTOT)	0.04

Where:
ZMAX = Maximum lake depth,
VDI = volume development index (bottom configuration),
WSIA = annual wind stress index (lake orientation to prevailing winds),
Z = mean depth,
FLTOT = annual loading rate.
VOL = lake volume, and
TIME = hydraulic residence time,

correlated with the salinity factor. WSIA is a shape factor that weights the configuration and orientation of a lake with the seasonal distribution of wind vectors and, independent of lake area, estimates the relative quantity of energy transferred to the water column because of wind-induced mixing.

The effect of VDI on the salinity factor is difficult to interpret in a manner consistent with other parameters in the model. VDI is a crude indication of the overall bottom configuration and represents the ratio of lake volume to that of a cone with basal surface area (corresponding to lake surface area) and height ZMAX. Consequently, VDI reduces to:

$$VDI = \frac{3\bar{z}}{ZMAX} \qquad (3)$$

The relationship of VDI in the stepwise regression model implies that for reclaimed lakes, a conical basin configuration (i.e., low VDI) results in higher ionic content than more developed, teacup-shaped basins. This aspect of the model may reflect greater amounts of calcareous bedrock exposure in poorly developed lakes.

SUMMARY

From a water quality perspective, two factors were separated for further evaluation. One factor represents lake trophic state indicators (Total N, Total P) and another represented water column inorganic nitrogen concentration. Maximum lake depth was negatively correlated with trophic state indicators and was also the most important morphometric relationship. Interpretation of this relationship would indicate that as maximum depth increased, concentrations of indicators declined. Sedimentation of detrital material may be the process involved in removing nutrients from the trophogenic or biologically active zone. In contrast, mean depth was positively correlated with trophic state indicators. This relationship may be confusing at first; however, it does present a situation where deep holes can be a vital component of lake design while keeping the overall mean depth in line with natural or unmined lakes. Sediments throughout the basins for each reclaimed lake are periodically disturbed and resuspended, with net deposition and removal occurring only in deep holes. Irregular bottom contours, consisting of intermittent deep holes, require greater wind energy to induce mixing than smooth bottom lakes. Since reclaimed phosphate pit lakes are either mesotrophic or eutrophic, the reduction of surplus nutrients by sedimentation to deep water zones with minimal recirculation potential can be a major factor in lake design.

The inorganic nitrogen factor provided indications that reclaimed lake water quality is in a transitional state, dependent upon inorganic nitrogen from allochthonous sources. This factor was negatively correlated with lake mean depth. Since phosphorus is in sufficient supply within these lakes, nitrogen supply is critical to maintain high productivity levels. Increasing mean depth would remove nitrogen from the trophogenic zone. This factor was also strongly correlated with lake residence time (+) and hydraulic loading rate (+).

The strongest statistical relationship was with the water column ionic content factor. These results indicated that penetration of the lake basin into the calcareous bedrock underlying the surficial unconsolidated layer of sands controls the ionic character of the water column. Both mean depth and orientation to prevailing wind stress were primary lake design features relative to water column ionic strength. Maximum depth was also significant, indicating the importance of deep holes as traps or sinks for detrital or easily weathered material. Since ionic content is governed by internal processes, lake orientation to wind can aid in redistribution and recirculation of ionic constituents.

The overall effect of a particular morphometric or hydrologic design variable can be evaluated relative to other variables by considering its significance in each factor relationship (as determined by its F value), the amount of overall variability accounted for by each factor, as well as the strength or explanatory power of each model given by the coefficient of determination.

This approach indicates overall data variability appears most strongly influenced by the two depth variables, ZMAX and Z, followed by lake volume and volume development. Lake hydrology embodied as the hydraulic loading rate and residence time was somewhat less important, as was lake orientation (WSIA). The remaining morphometric/hydrologic variables were not particularly effective in accounting for overall variability in reclaimed lakes.

A final word of caution is perhaps in order. Implications of cause and effect have been intended to account for the correlations between the various morphometric/hydrologic variables and the chemical attributes of the reclaimed lakes. For example, supporting mechanisms have been hypothesized to account for the statistical relationship between ZMAX and the lake trophic state and salinity factors. However, correlations between variables do not confirm cause and effect, and the possibility that ZMAX may be correlated with some other (unmeasured) parameter that is essentially the true cause of the observed effect cannot be discounted. Further research is necessary to confirm the cause-and-effect implication.

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THE APPLICATION OF QUAL-II TO AID RESOURCE ALLOCATION ON THE RIVER BLACKWATER, ENGLAND.

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by

Bob Crabtree, Ian Cluckie, Paul Crockett and Chris Forster; Department of Civil Engineering, University of Birmingham, Birmingham B15 2TT U.K.

ABSTRACT

The River Loddon is a tributary of the River Thames. It has a deterioration in water quality downstream of the confluence with its major tributary, the River Blackwater. The Blackwater catchment is densely populated and the river is of poor quality due to a high proportion of treated effluent. Qual-II has been applied to this system as a forecasting tool to investigate any improvements in the quality of the River Loddon that may be achieved by altering the allocation of effluent treatment resources within the Blackwater Catchment. In particular, the effects of a proposal to extend an existing effluent treatment works were evaluated.

INTRODUCTION

The daily domestic consumption of water in England and Wales is some $16 \times 10^6 m^3$. The majority of this is eventually discharged to inland rivers and streams, the total length of which is some 38 000 km. In addition, about $20 \times 10^6 m^3$ is abstracted directly from these rivers by industry, used and then returned. Some of these waters will have become contaminated during their use and it is therefore necessary for them to undergo treatment before they can be discharged. Failure to do this can result in serious changes in the chemical composition of the river water, causing deterioration in the river ecology possibly coupled with a significant degree of oxygen depletion.

Rivers within the U.K. have to accept a significant load of oxygen demanding pollutants. However, as long as the load on any particular reach is not excessive, the river can assimilate it. Discharges of this type which have occurred on a regular, continuous basis for sometime are likely to have caused a change in the downstream ecology, relative to the time before the discharge. As long as the load is not increased, this "new" downstream ecology will be stable and, in a lot of cases, will be acceptable. However there is little point in knowing what pollutants are present, either in an effluent discharge or in the river itself, or knowing what effect those pollutants may have on the stream biota, if their addition to natural waters cannot be controlled. Controlled that is, in such a way as to reduce their impact to a level acceptable for all the uses required for the river, which may include transport, amenity, and potable and industrial water supply, as well as waste disposal. This is particularly true if the river is to be used to its maximum economic potential by paying full regard to to its ability for self purification. Models, properly applied, can help to optimise the use of rivers.

River quality models are commonly used to forecast dissolved oxygen levels (DO) at a point in a river system resulting from the interaction of the physical and biological assimilative processes of oxygen demand. These models can then be used to predict the effects of proposed water quality control strategies on water quality and use. Modelling can therefore be used as an aid to quality management decision making and resource allocation to achieve designated levels of quality, termed river quality objectives (RQO) in the U.K.

There are two general types of model that have been applied to river systems; stochastic and deterministic. Seven out of the ten U.K. Regional Water Authorities (RWA) have at some time used a form of deterministic model for predicting river water quality (Russell and Pescod, 1981) or assessing the impact of effluent discharges on downstream river quality. The models used are typically river specific derivatives of the Water Resource Centre river quality model, developed by Knowles and Wakeford (1978). This evaluates downstream quality changes of conservative and non-conservative substances in a reach by reach mass-balance basis and incorporates the effects of mixing, dispersion and biological processes. It is no longer widely used and it is generally only applied to specific problem rivers. Compared with North America, the use of deterministic river quality models for water quality management in the U.K. is in its infancy. The RWA control all aspects of flood control, water supply, drainage, sewerage, effluent treatment and river pollution management. They are large, catchment-based public bodies which are charged with conducting their activities to achieve a satisfactory balance between the quality of the services provided and the resulting costs borne by the consumer. The main dischargers of sewage effluent to the rivers are the RWA who operate the sewage works. These treat domestic sewage and much of the industrial trade effluent which is allowed to be discharged to the public sewers. Therefore the main method of controlling river quality is by controlling the discharges of effluent to the river. This can lead to a situation, depending on local circumstances, where the RWA is both a "poacher" and a "gamekeeper", causing and managing pollution.

Historically, effluent discharge standards (consents) have been a legally binding value and have tended to be based on the 1912 Royal Commission standard of 30 mgl^{-1} suspended solids (SS) and 20 mgl^{-1} BOD, without due consideration of the effect on the receiving water course. This ignores the fact that the 30:20 standard was based on the quality of effluent produced by a well operated biological filtration plant and assumed that the discharge was to a water course with a dilution factor of at least 8:1. The use of a blanket 30:20 discharge standard may result in either:

1) a consent which is unnecessarily stringent in some circumstances or not stringent enough in others; or

2) a consent which is inconsistent with existing treatment facilities or with the cost of improvement.

To avoid either river pollution or undue capital cost, the National Water Council (1977) recommended that effluent discharge consents should take account of local circumstances and should be based on the desired downstream river quality, the RQO. While discharge consent conditions would still be fixed figures which could not legally be exceeded, it was not expected that RWA or private individuals would launch prosecutions where occasional samples exceeded the consent conditions. This was with the proviso that samples exceeding the consent were not greater than the variability to be expected from a well managed system.

In 1977 the assessment of river and effluent quality in the U.K. was placed in a probabilistic framework. A new classification of river water quality, shown in simplified form in Table 1, was adopted. This was based on quality criteria (RQO) in terms of 95 percentile class limits. The 95 percentile is a sample based statistic and is the value of the river quality that could be expected to be achieved in 95 percent of samples taken. The use of probabilistic class limiting criteria for river quality and effluent discharges reflects the underlying stochastic nature of river quality and flow processes and permits the statistical treatment of quality data within such a probabilistic framework (Cluckie and Forster, 1982).

In 1978, the National Water Council (NWC, 1978) made recommendations to ease the implementation of the 1974 Control of Pollution Act, part II whereby there would be greater public accountability by the RWA for the management of pollution control. The recommendations allowed the resetting of consents to suit local environment, financial and technical conditions prior to the implementation of the new act.

The new consents are set in terms of interim consents (to maintain present levels of river quality) and long term consents (to achieve the designated RQO). The long term consents can be set by optimising river pollution control by whole catchment based river quality models in which, for a river system subdivided by RQO, full use of river self-purification could be made between effluent discharges. Therefore the correct consents

TABLE 1. SIMPLIFIED NATIONAL WATER COUNCIL (1977) CLASSIFICATION OF RIVER QUALITY

River Class	Class limiting quality criteria (95 percentile)	Comments
1A	DO greater than 80% saturation BOD not greater than 3 mgl^{-1} Ammonia not greater than 0.4 mg^{-1}	Suitable for potable supply game fisheries and high amenity value.
1B	DO greater than 60% saturation BOD not greater than 5 mgl^{-1} Ammonia not greater than 0.9 $mg\ell^{-1}$	less high quality than 1A but visible evidence of pollution should be absent.
2	DO greater than 40% saturation BOD not greater than 9 mgl^{-1}	Moderate amenity value, capable of supporting coarse fisheries.
3	DO greater than 10% saturation BOD not greater than 17 mgl^{-1} not likely to be anaerobic	Visibly polluted, and likely to cause nuisance.
4	inferior to Class 3	grossly polluted and likely to cause nuisance.

are required to manage a river system to avoid undue treatment costs or downstream pollution with the risk of prosecution. The simplest method for calculating consents is a simple mass balance of conservative substances, which assumes instantaneous mixing of a single effluent entering a river, shown in Figure 1.

This has the form:

$$C_1 = \frac{(Q_o + Q_1)C_2 - Q_o C_o}{Q_1}$$

where Q_o = 5% upstream flow (95% exceedence flow)

C_o = mean concentration upstream of discharge

Q_1 = mean effluent flow

C_1 = effluent discharge consent

C_2 = 95% downstream class limiting concentration

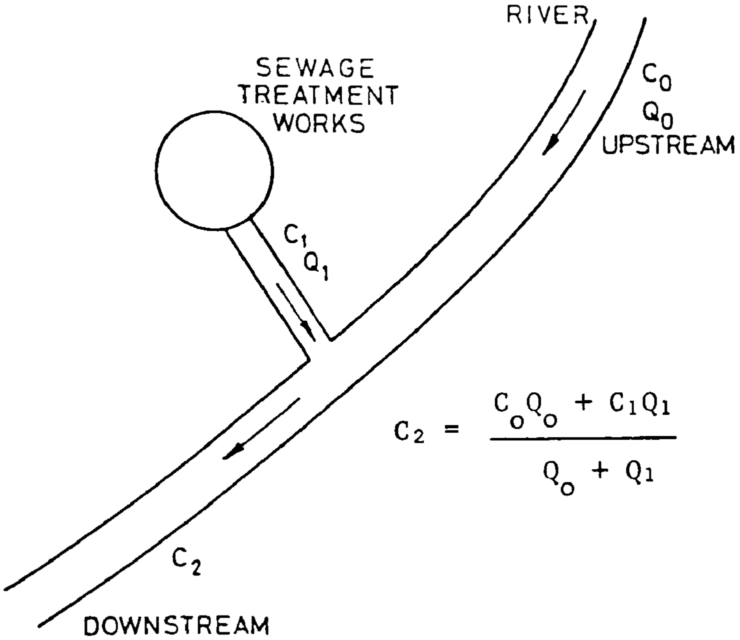


Figure 1. Simple mass balance mixing for an effluent discharge

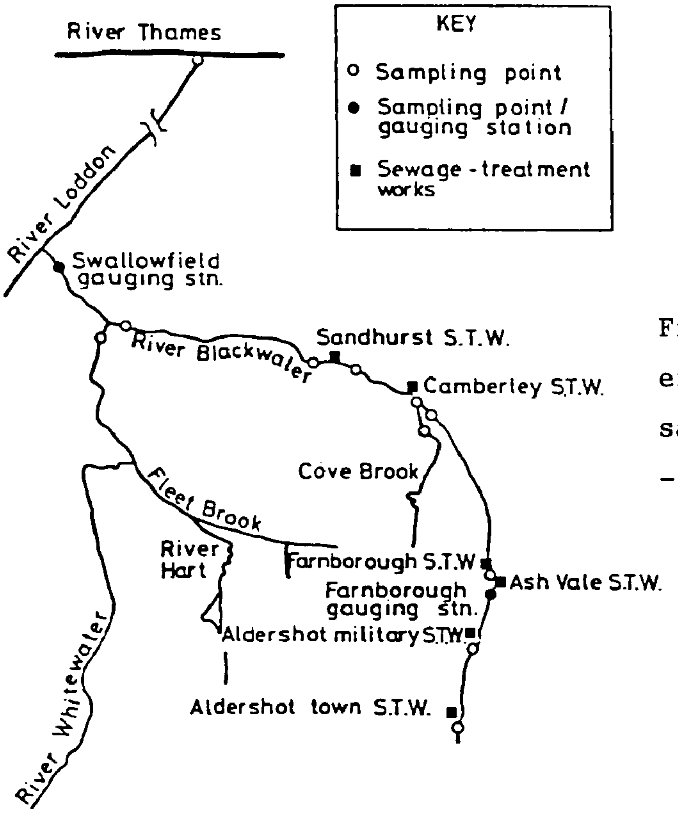


Figure 2 Location of effluent discharges and river sampling points on the Loddon - Blackwater river system.

There has been much critisism of this equation. At best it results in a conservative value of C_1 which is an unknown statistic, not a 95 percentile value, as recommended, for consent setting. Statistical mixing techniques (Cluckie and Forster, 1982) can be used to give a more robust statistical value, but this can only be applied to single point discharges and takes no account of the self-purification processes acting in the river. To do this for a number of discharges on a river system some form of catchment based river quality model is required. The purpose of this study is to apply QUAL-II to a pollution control problem on a U.K. river in an attempt to optimise the allocation of sewage treatment facilities on that river system.

THE BLACKWATER-LODDON RIVER SYSTEM

The river Loddon is a tributary of the river Thames, with its catchment area to the south west of London. The river Blackwater is the main tributary of the Loddon, shown in Figure 2. Above its confluence with the Blackwater the Loddon has a quality class IB, but class 2 below the confluence (RQO = 1B) due to a high BOD load from the river Blackwater (Table 2).

TABLE 2 RIVER QUALITY FLOW AT THE CONFLUENCE OF THE RIVERS LODDON AND BLACKWATER. 1981 - 1982

	Mean Flow $m^3 s^{-1}$	D.O. mgl^{-1}	BOD mgl^{-1}	NH_4-N mgl^{-1}	NO_3-N mgl^{-1}	CLASS*
Loddon[1]	2.44	10.0	1.6	0.1	7.5	1B
Blackwater	3.38	9.2	2.4	0.3	6.8	2
Loddon[2]	5.82	8.9	2.2	0.2	7.2	2

1 above confluence

2 below confluence

* National Water Council 1977 River Quality Classification results as means for the two year period.

Class limiting criterion is BOD

The quality of the Blackwater is the problem in the system. The river Blackwater flows through an urbanised catchment area underlain by Tertiary clays and sands, and has a naturally poor water quality and is designated as quality class 2 (RQO = 2). The headwaters of the Blackwater contain several old, overloaded sewage treatment works and the river carries a high proportion of treated sewage effluent, which can be as high as 70% at times of low flow.

TABLE 3 RIVER QUALITY OBJECTIVES AND ACHIEVEMENTS 1976 - 82

		River Quality Class	
	Reach length (km)	Objective	Achievement
River Loddon			
Source to Basingstoke S.T.W.	7.8	1A	1B
Basingstoke S.T.W. to River Blackwater	18.2	1B	1B
Blackwater to Confluence Thames	19.2	1B	2
River Blackwater			
Source to Aldershot S.T.W	4.5	2	2
Aldershot S.T.W. to Cove Brook	11.7	2	3
Cove Brook to Confluence Loddon	19.5	2	3

Table 3 shows that despite a RQO of class 2, the Blackwater below Farnborough sewage works is a class 3 river until the effluent load is diluted below the confluence with the R. Whitewater, which is class 1B. This current situation, with a large portion of the Blackwater in class 3, is itself undesirable for Thames Water Authority (TWA). However, its effect on the Loddon, shown in Figure 3, is the major problem as this source could be used for potable water supply.

The analysis of the sewage treatment works performance data, tabulated in Table 4, shows that with the exception of Camberley, all the works achieve compliance with their consents. However these are short term or interim consents, set to match the current works performance, not the desired downstream RQO. The consents for Ashvale, Farnborough Camberley and Sandhurst all indicate poor performance due to overloading and delapidation of the treatment plants. An additional problem for water quality management on the Blackwater is that of Aldershot Military sewage treatment works. This works is operated by the Government, it has no consent and its discharge quality and quantity is unknown. A simple mass balance suggests that the mean flow is around 7 Ml per day, with an operational performance equivalent to a consent of 30/20/5.

To improve the quality of the Blackwater and the Loddon to achieve

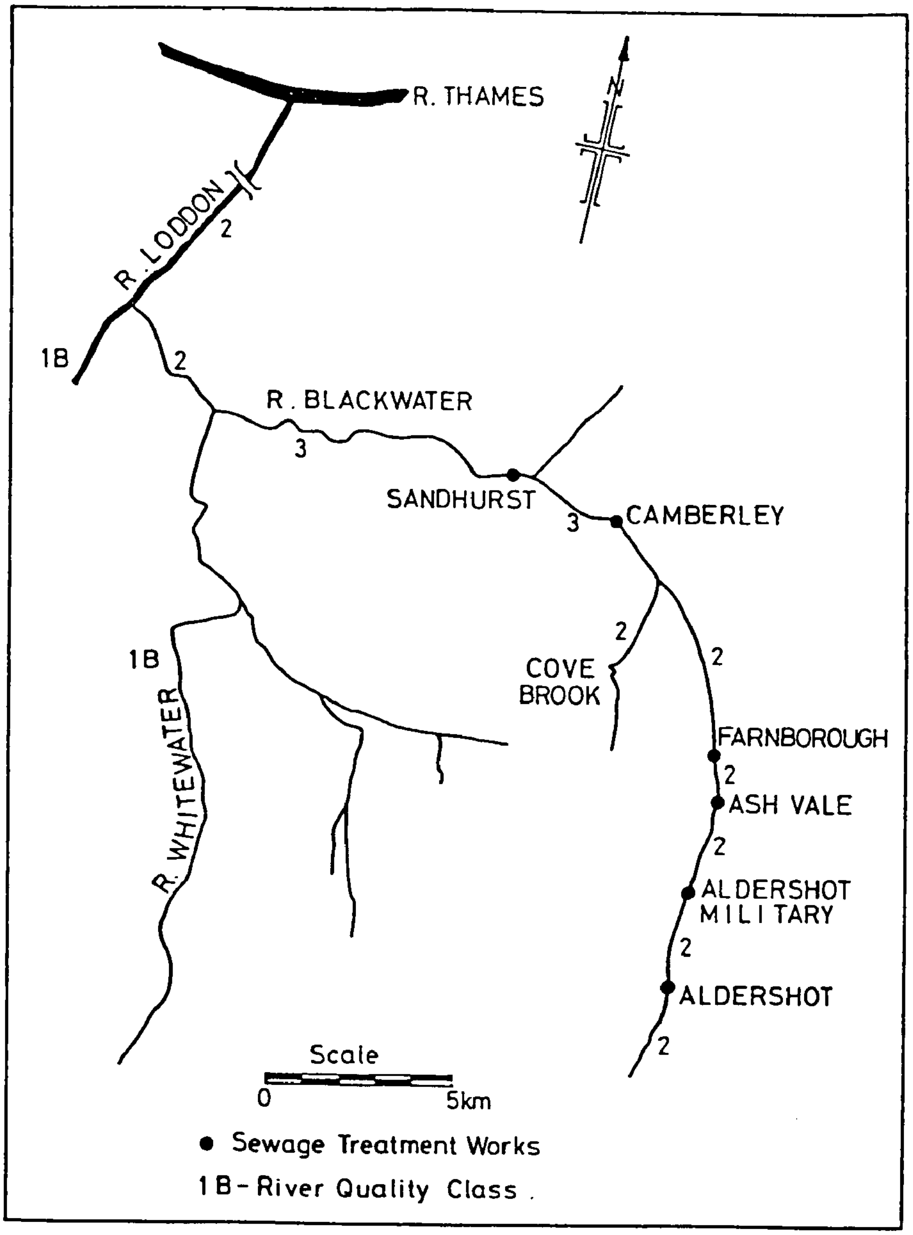


Figure 3 Current river quality classification for the Loddon-Blackwater river system, based on mean annual data, 1973 - 1981

TABLE 4 RIVER LODDON - BLACKWATER SYSTEM: SEWAGE TREATMENT WORKS PERFORMANCE

Works	Stream	Consent SS/BOD/AMM	% Compliance with consent
Basingstoke	Loddon	15/10/10	96
Aldershot	Blackwater	20/12/3	95
Ashvale	Blackwater	40/12/15	99
Farnborough	Blackwater	40/20/15	96
Camberley	Blackwater	20/25/5	92
Sandhurst	Blackwater	25/9/12	95

the designated RQO, TWA have proposed two improved management schemes for sewage treatment on the river Blackwater. The two schemes are designed for the river system to achieve its RQOs with larger flows resulting from the increased urban and industrial development predicted for 2020 A.D. Each of the schemes, shown in Figure 4 has three phases of development to spread capital expenditure. Both assume no changes at Aldershot Town and Aldershot Military STW, with present performance and flows maintained. A capital expenditure programme at Ashvale is already committed and this will take flows upto 9.0 Ml/day with a consent of 10/10/3 by using a new Carrousel treatment system (Forster, 1980).

The ultimate aim of scheme B was to close Farnborough, Camberley and Sandhurst works and build a complete new works at a new site. This proposal has been abandoned due to high cost. Therefore quality modelling has been limited to forecasting the effects of Scheme A. This proposed the eventual closure of Farnborough and Sandhurst and the diversion of flows to Camberley. At Camberley, land is available for expansion at the present site. In phase III the works would be expanded to treat the increased flow but only to present levels of performance. The present works is overloaded and produces a poor quality effluent.

WATER QUALITY MODEL APPLICATION

Water quality problems on the river Blackwater were originally investigated by Casapieri et al (1978) using a version of the W.R.C. river quality model (Knowles and Wakeford, 1978). This, "Blackwater" model, is river specific and was developed for TWA to predict the possible effects of population growth. The deterministic model is one dimensional and operates in a steady state mode. A plug-flow of water passing down the river is simulated by splitting the river into reaches, at each inflow a new reach is initiated. Non-point sources must be entered as point inflows, thereby

Figure 4. Thames Water Authority effluent treatment management proposals

Sewage Effluent Flows to the River Blackwater (Ml day^{-1} mean)

WORKS	BASELINE FLOW	PROPOSAL A			PROPOSAL B		
		PHASE I	II	III	PHASE I	II	III
ALDERSHOT TOWN	10·5	10·5	10·5	10·5	10·5	10·5	10·5
ALDERSHOT MILITARY	7·0	7·0	7·0	7·0	7·0	7·0	7·0
ASH VALE	7·4	9·0	9·0	9·0	9·0	9·0	9·0
FARNBOROUGH	7·5	8·4			8·4		
CAMBERLEY	16·4	22·0	41·2	55·4	13·2	13·2	
SANDHURST	11·8	16·1	14·2		13·9	14·7	
NEW WORKS					11·0	27·5	55·4

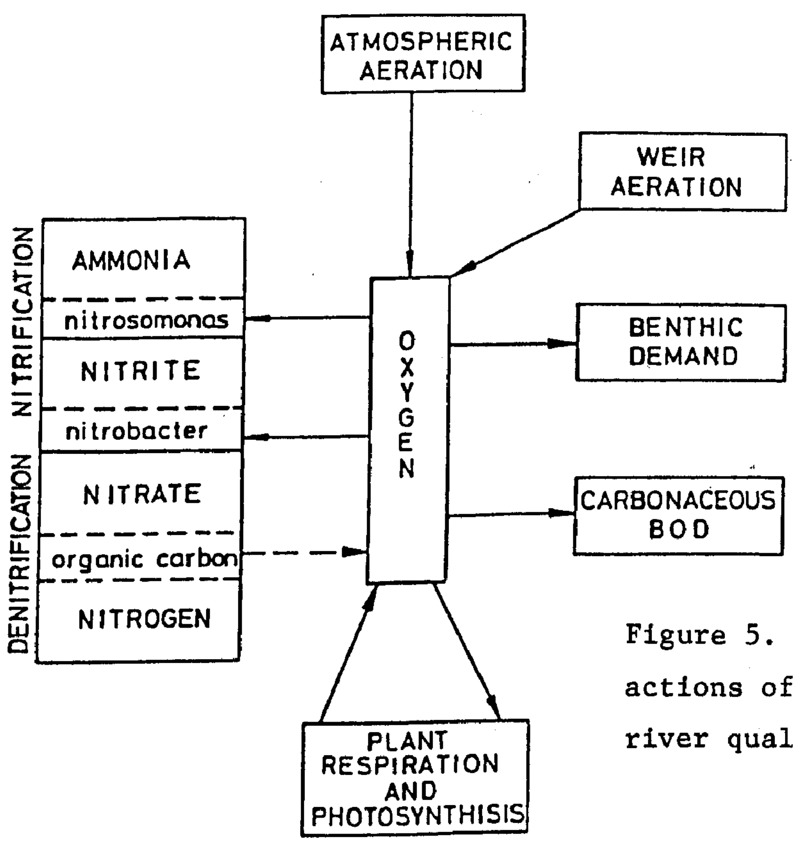


Figure 5. Major biochemical interactions of the W.R.C. Blackwater river quality model.

starting a new reach. The water quality constituents simulated are DO, BOD, ammonia (NH_4-N) and nitrate (NO_3-N). Longitudinal dispersion and river temperature are not modelled. The rate of nitrification is controlled by the concentration of Nitrosomonas bacteria. The model input requires user - calculated hydraulic relationships for each reach, for example, depth, time of travel and channel cross-section. Also, relative plant density for each reach must be supplied. The model is based on fieldwork derived rate constants and constants. Mathematical solution is by numerical integration of complete differential equations. The main interactions of the model are shown schematically in Figure 5. The model has not been used since the initial study and TWA have now implemented a general purpose stochastic river quality model.

As a long term planning tool, the Blackwater model was run using triannual mean data to predict mean annual river conditions. In this study the NCASI (1980) version of QUAL-II SEMCOG (EPA, 1981) was applied to the Loddon-Blackwater system using the same calibration data (1973-1975 triannual mean) as used for the Blackwater model. Wherever possible, the same values for constants and rate constants were used in both models. The main differences between the two models are shown in Table 5.

TABLE 5 WATER QUALITY MODEL COMPARISON

1) W.R.C. Blackwater Model:

Deterministic, steady state

input: 17 rate constants

15 physical constants and parameters values

output: Dissolved Oxygen, BOD, Ammonia, Nitrate,

- Specific to River Blackwater.

2) U.S. E.P.A. Qual-II Model:

Deterministic, steady state/pseudo dynamic

input: 28 constants and river identification parameters

16 parameter values

output: 8 non conservative substances

3 conservative substances

+ temperature

- Generally applicable to any river system

QUAL-II is a non specific model and has a complex representation of biochemical interactions. The main differences in the biochemical modelling by the two models are:

1) the use of algae by QUAL-II and plant density by the Blackwater model, as biological factors influencing DO concentrations.

2) the Blackwater model uses a single stage nitrification mechanism controlled by the concentration of Nitrosomonas bacteria. QUAL-II uses a two stage feedback mechanism of algal growth to influence the conversion rates from amonia to nitrite and nitrite to nitrite.

The parameter estimates used in applying QUAL-II are shown in Table 6. The Blackwater has significant weed growth during the summer and it was possible that this might cause problems with QUAL-II. QUAL-II is also usually used to predict specific situations, not general forecasts based on means as input data.

TABLE 6 PARAMETER ESTIMATES USED IN BLACKWATER/LODDON CATCHMENT SIMULATION

Parameters	Value/Comments
Reaeration Rate	Tsivoglou and Wallace, 1972 reach coeff 1.3
Longitudinal Dispersion	Elder (1959)
Decay/Oxidation Rates (1 day^{-1})	
BOD-U	0.15-0.25
Ammonia N	0.35
Nitrate N	1.0
Algal Parameters	
Maximum Growth Rate	2.5 1 day^{-1}
Respiration Rate	.1 1 day^{-1}
Settling Velocity	.15 m day^{-1}
Chlorophyll Content	.050 μg Chl-a mg^{-1} Algae
P Content	.012 mg P mg^{-1} Algae
N Content	.085 mg N mg^{-1} Algae
Light Extinction	38.0 m2 g^{-1}Chl-a
Half-Saturation Constants	
Algal P Uptake	.04 mg m^{-3}
Algal N Uptake	.3 mg m^{-3}

Benthic Oxygen Fluxes gm^{-2} day^{-1} - 0.0

Both models were calibrated using triannual data for 1973-1975 and then verified and recalibrated using data for 1975-1978. They were then used to predict mean annual quality for 1981 and the results were compared to the observed mean river quality. The comparisons for DO, BOD and NH_4-N are shown in Figure 6. To assess the goodness-of-fit of the models to the observed data, the weighted root mean square error (WRMSE) statistic (Anglian Water Authority, 1979) was assessed for each of the four modelled variables, where:-

$$\text{WRMSE} = \frac{\sum_{i=1}^{N} W_i (O_i - C_i)^2}{N}$$

O_i is the observed concentration at the i^{th} sampling point

C_i is the calculated concentration

N is the total number of sampling points

W_i is a weighting factor, the reciprocal of the standard deviation of the observed concentrations.

The WRMSE statistic is non-dimensional and allows a comparison of results for different variables. The smaller the value of the WRMSE than statistic then the closer is the fit between observed and calculated results. Table 7 shows a comparison of the results.

TABLE 7 WRMSE STATISTIC VALUES

Variable	Model	
	QUAL-II	Blackwater
DO	0.47	0.80
BOD	0.35	0.49
NH_4-N	0.42	0.41
NO_3-N	0.82	1.23

from 1981 mean annual results.

Neither model gave a perfect prediction. The results for QUAL-II showed that BOD and NH_4-N were the 'best' modelled variables, followed by DO and NO_3-N. In selecting a model for any specific situation, it is important to use the simplest model that will yield adequate results, that is, fitting the model to the problem, not the problem to the model. In this study the two models differ markedly in their complexity. However the simpler Blackwater model produces poorer results than QUAL-II. The major difficulty in applying either model is the availability of sufficient hydraulic data.

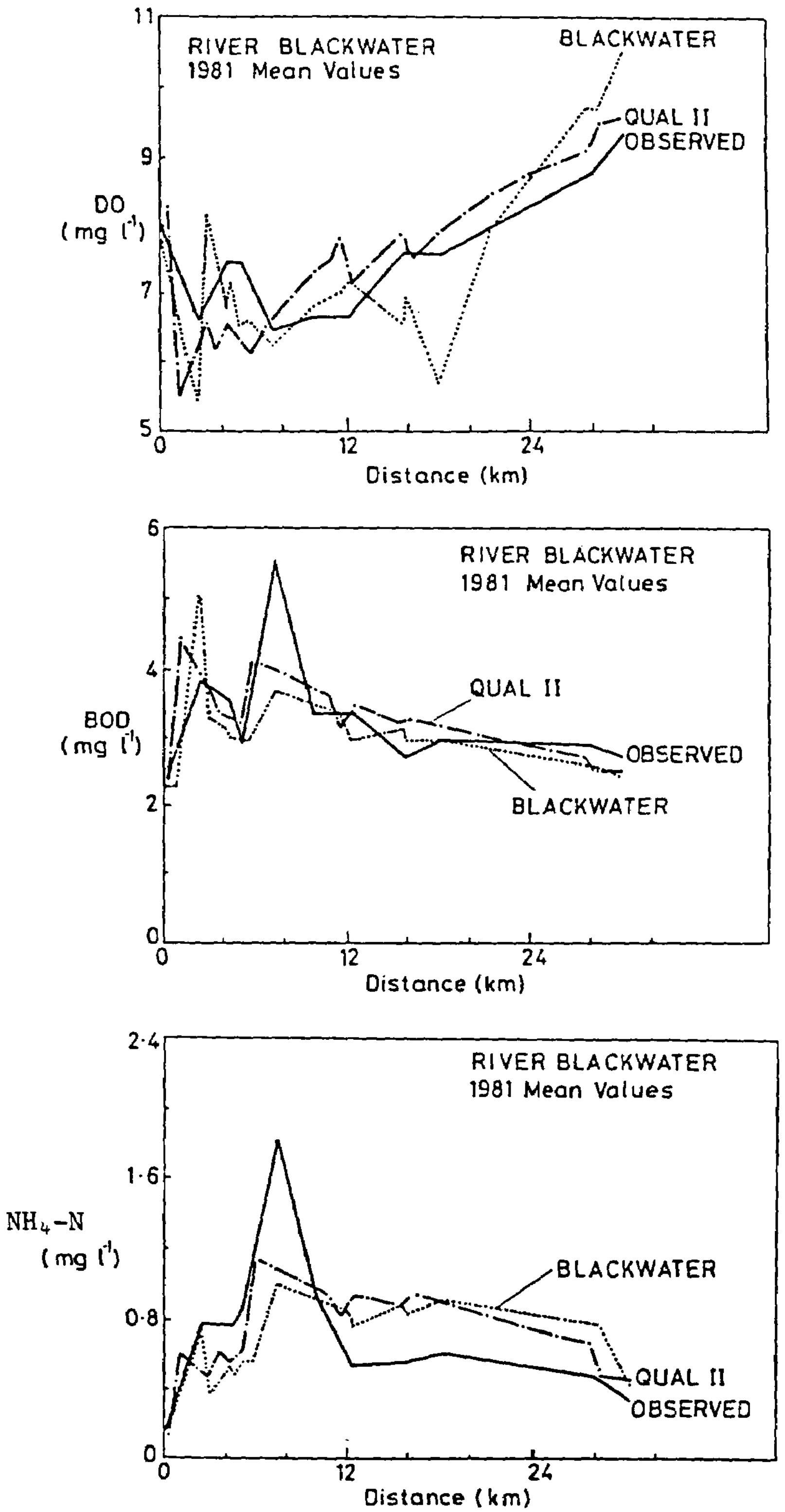


Figure 6. Comparison of Water Quality Model predictions

Also only routine monitoring quality data was available.

Despite these data problems, the comparison of the models showed that QUAL-II could be applied to the Blackwater-Loddon system to predict mean annual river quality conditions.

The QUAL-II model was then recalibrated using 1981 data and reverified against 1982 data. The observed and predicted results are shown in Figure. 7. Although these were better than for the model comparison, NH_4-N was still the worst modelled variable. The predicted mean values for 1982, shown in Figure 8, indicate a slight improvement in the river quality since the 1970s. DO is not a class limiting parameter, however the river Blackwater is in class 2 above its confluence with the river Whitewater due to high values for BOD and NH_4-N. There are no class 3 reaches on the Blackwater but the river Loddon is still class 2, not 1B below its confluence with the Blackwater. BOD is the limiting parameter.

FORECASTING THE EFFECTS OF CHANGED TREATMENT STRATEGIES

In modelling the effects of the scheme A proposals for changes in the effluent treatment strategy on the river Blackwater, shown in Figure 4, it was decided to attempt to obtain results within the probabalistic framework of the consents and RQO. To do this QUAL-II was run on a worst case basis. Effluent discharges were set to mean flows and the quality of each effluent was taken as its 95 percentile (i.e. consent) quality. 95 percentile values for river quality and 95 percent exceedence river flows were modelled. While it is recognised that this worst case modelling procedure will give results that are not 95 percentile values, the results will be highly conservative and will probably overestimate the true 95 percentile values. Therefore they can be used as guidelines for assessing the predicted river quality in terms of the 95 percentile class limiting criteria used to test for compliance with the set RQO standards. This is an initial crude attempt to use QUAL-II with stochastic input variables to predict river quality within the probabalistic framework of consents and RQO used in the U.K.

The results of modelling the worst case situation for phase I of the scheme are shown in Figures 9 and 10. In this phase, effluent discharges would be increased but there would be no changes to the treatment facilities. The results showed that the quality of the Blackwater and Loddon would deteriorate. The main limiting variable would become NH_4-N. Parts of the Blackwater would become class 3. This would be worse than the present situation, therefore phase I proposals are not acceptable. Similarly phase II was also assumed to be unacceptable and was not modelled. Phase III proposals involve closing Farnborough and Sandhurst works and diverting the flows to an enlarged works at Camberley. However, this works would maintain its present consent. The results of this proposal would be worse than those for phase I, as shown in Figures 9 and 10. Phase III proposals cause a high input of BOD and NH_4-N at a single point in the Blackwater resulting in class 3 conditions downstream.

The only possible solution to the problem was to reduce the pollutant

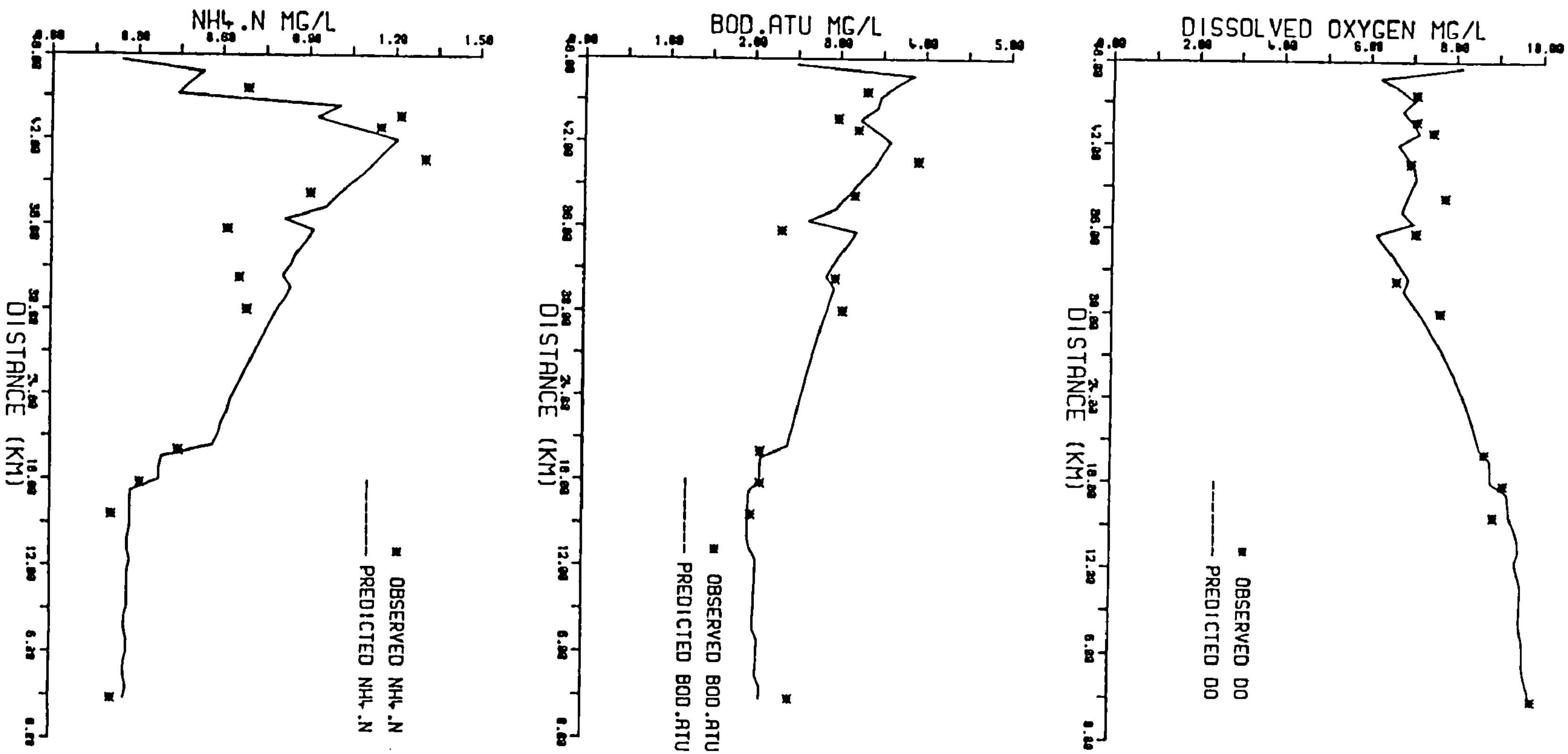


Figure 7. Comparison between observed and QUAL-II predicted mean annual river quality on the Loddon-Blackwater river system for 1982.

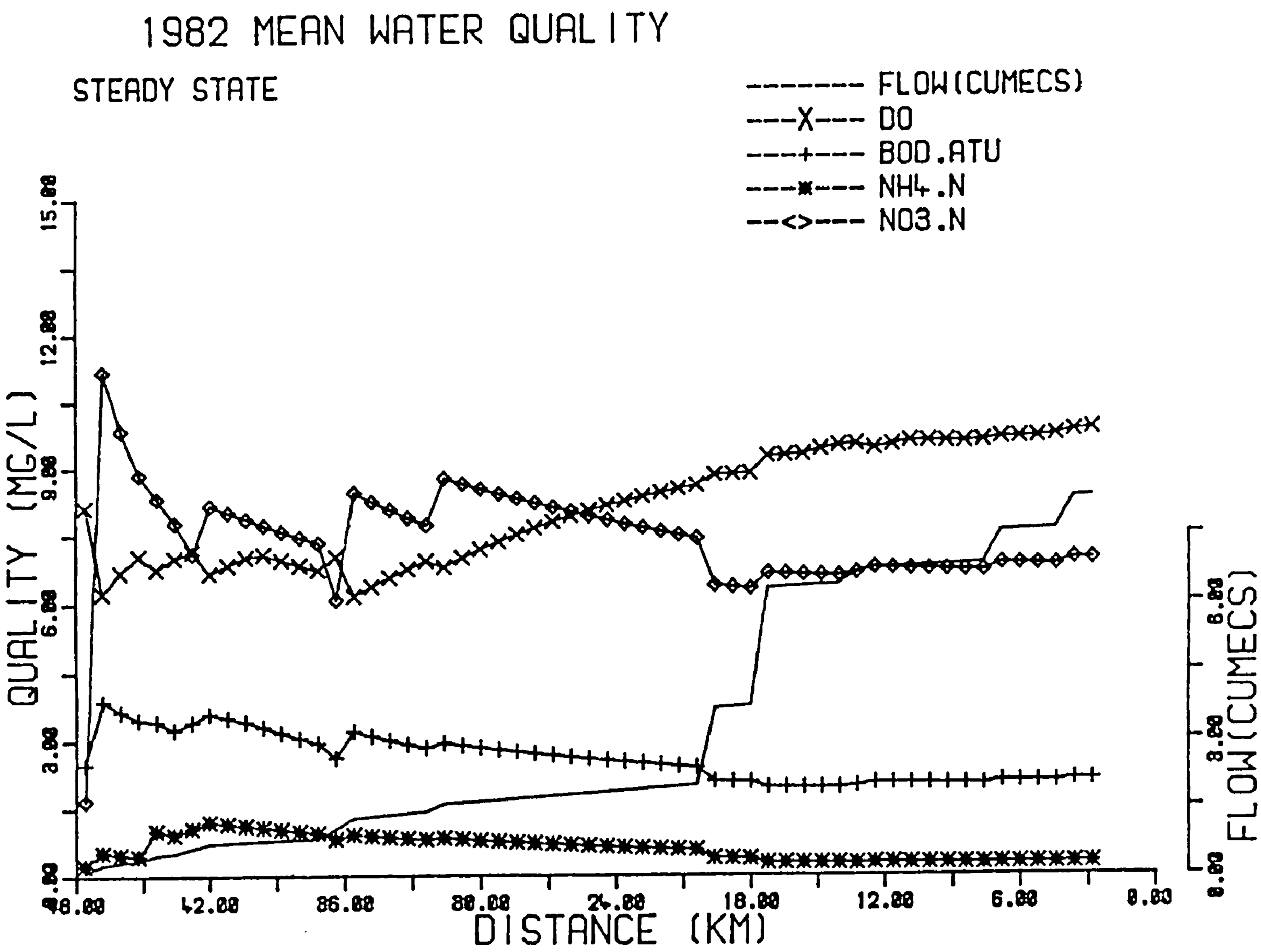


Figure 8. QUAL-II predicted river quality for the Loddon-Blackwater river system

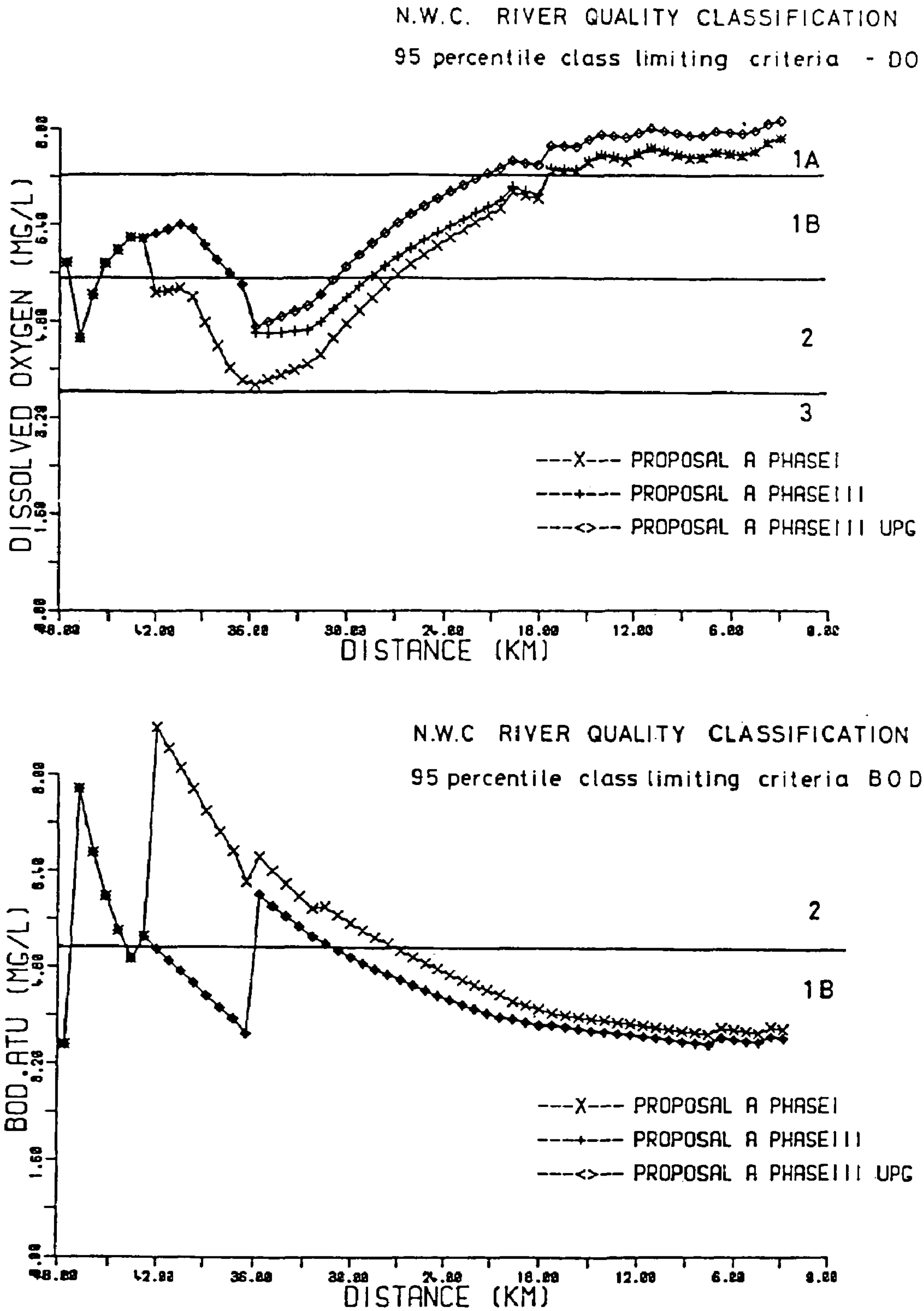
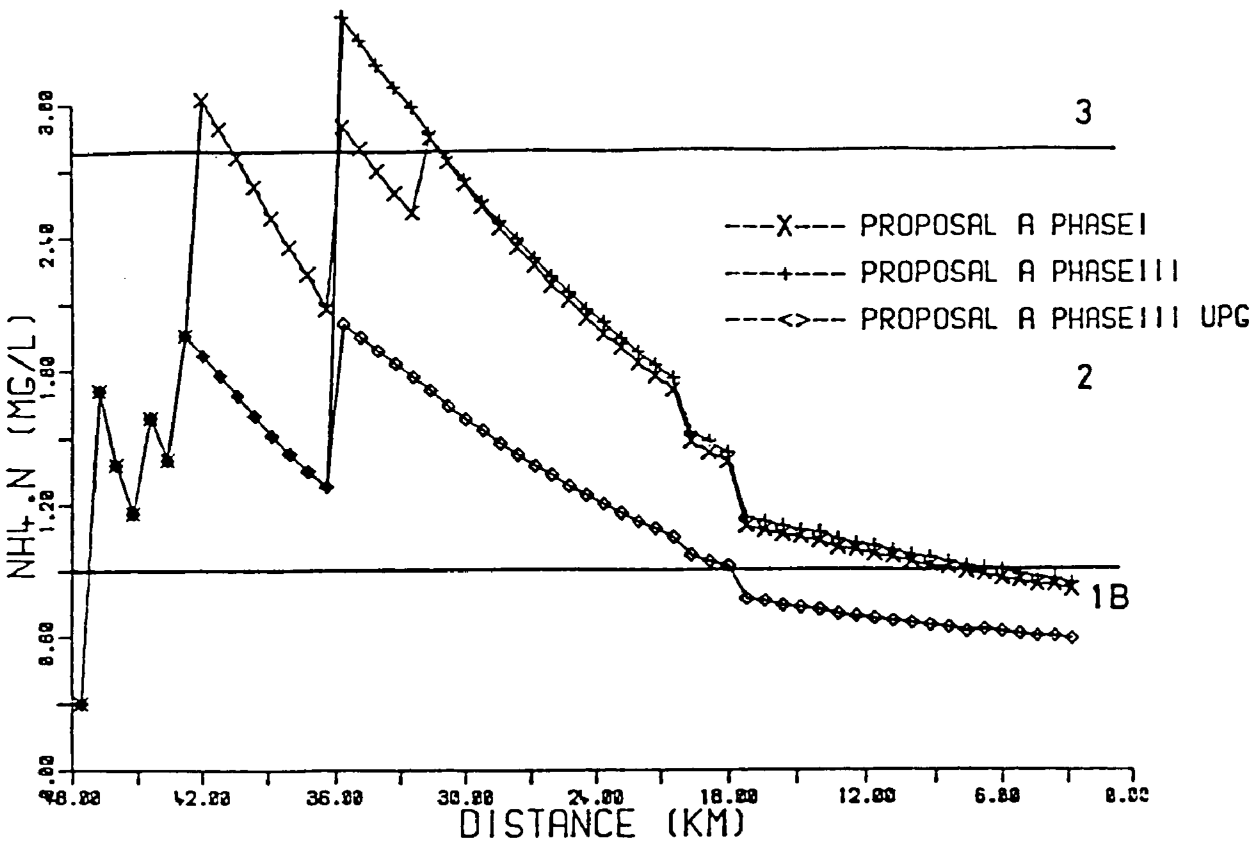


Figure 9. Predicted worst case effects of effluent management schemes on river quality -DO and BOD



N.W.C. RIVER QUALITY CLASSIFICATION

95 percentile class limiting criteria $NH_4 \cdot N$

Figure 10. Predicted worst case effects of effluent management schemes on river quality NH_4-N

load entering the river Blackwater. This could not be done using the existing treatment facilities and the scheme A proposals. Scheme B, to build a complete new works at a new location, was not feasable. Therefore a new scheme had to be proposed. Land for expansion is available at Camberley and pollutants from Camberley have time to be assimilated in the river before entering the river Loddon. Therefore it was decided to base the new proposals on upgrading the existing plant at Camberley. Farnborough and Sandhurst would be closed and the flows diverted to Camberley. At Camberley, 25% of the existing flow, and the diverted flow plus the future forecasted flow increment would be treated by a new Carrousel activated sludge plant. This would enable the present biological filtration plant to be retained and to perform satisfactorily without overloading. The new Camberley plant would be similar to the new system under construction at Ashvale and would be expected to produce a similar strength effluent (Forster, 1980). The new works would have a consent of 10/10/3 instead of the present consent of 20/25/5.

The results of modelling this proposal (scheme A, phase III with upgrading) as a worst case situation are shown in Figures 9, 10 and 11. With this proposal all the RQO could be complied with. The river Blackwater would become class 2 along all its length. Ammonia would be the class limiting variable, due to the poor effluent dilution. The main problem in the system would be overcome, the river Loddon would maintain its class 1B quality below the confluence with the Blackwater shown in Figure 12.

Thames Water Authority is now reconsidering the proposals for changes in the management of waste treatment facilities on the Blackwater-Loddon system in view of the effects predicted by applying QUAL-II.

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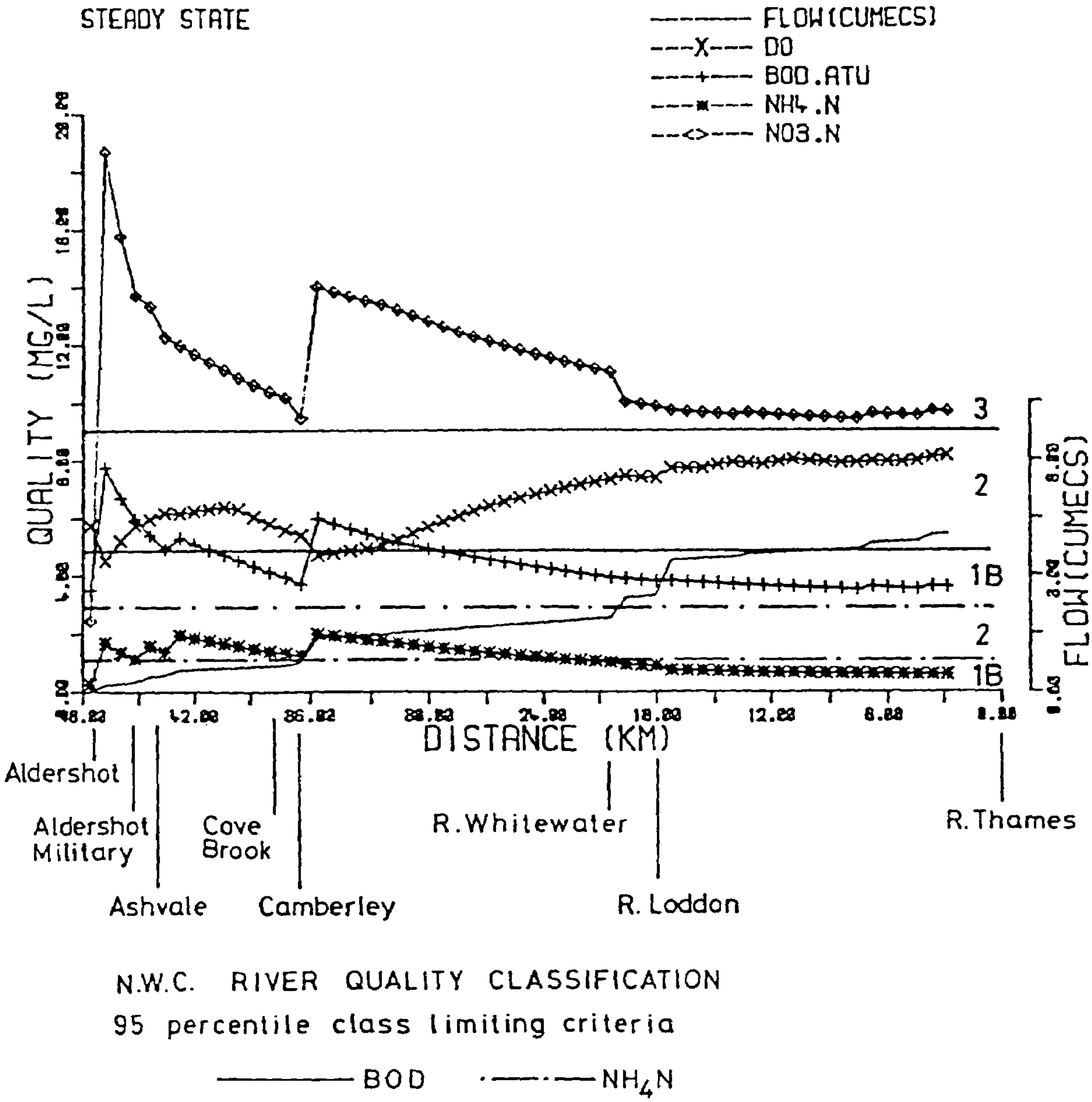

Figure 11. QUAL-II simulation of worst case river quality for the optimum effluent treatment management scheme

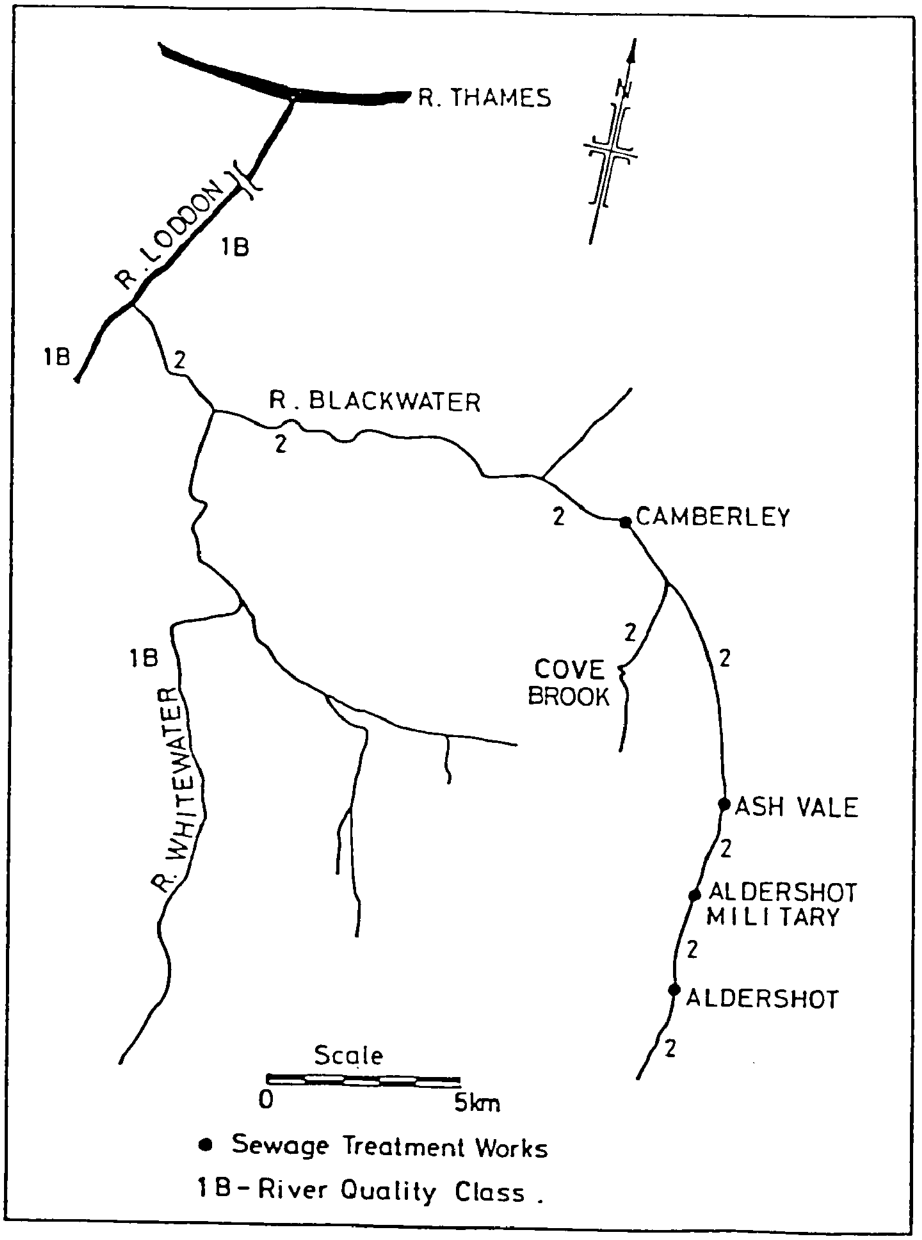

Figure 12. Predicted river quality classification for the Loddon-Blackwater river system with the optimum effluent treatment scheme.

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TECHNIQUES AND SOFTWARE FOR RESERVOIR EUTROPHICATION ASSESSMENT

by: William W. Walker, Jr.
Environmental Engineer
1127 Lowell Road
Concord, Massachusetts 01742

ABSTRACT

Results of a research project on empirical techniques for predicting eutrophication and related water quality conditions in reservoirs are summarized. The research is documented in a series of reports describing data base development(1), preliminary model testing(2), and model refinements(3). Using an extensive reservoir data base, previous models based primarily upon data from phosphorus-limited, northern, natural lakes have been tested and modified to improve their performance and generality in reservoir environments. The revised models account for algal growth limitation by phosphorus, nitrogen, light, and flushing rate. By considering nutrient sedimentation and transport mechanisms in a mass-balance framework, spatial variations in nutrients and related water quality conditions can be simulated. A manual and three computer programs have been developed to facilitate implementation of the models (4). The paper presents an overview of the research and software.

INTRODUCTION

Eutrophication can be defined as the nutrient enrichment of water bodies leading to an excessive production of organic materials by algae and/or aquatic plants. This process has several direct and indirect impacts on reservoir water quality and uses for water supply and recreation. Valid, practical assessment techniques for eutrophication are required to support reservoir water quality management efforts.

A four-phase research project has been undertaken to develop empirical modeling approaches for reservoir applications (1,2,3,4). The first phase involved the compilation and statistical summary of a data base describing morphometry, hydrology, and water quality conditions in 299 Corps of Engineer (CE) reservoirs (1). The second phase involved preliminary screening of existing models based upon the CE data base and an extensive list of model formulations compiled from the literature (2). The third phase involved model refinements and additional testing based upon independent lake and reservoir data sets (3). The fourth phase involved the development of an applications manual and supporting computer programs (4). Following are descriptions and illustrations of model structures and supporting software.

EMPIRICAL MODEL EVOLUTION

Eutrophication models can be broadly classified as theoretical or empirical. The former generally involve direct simulation of physical, chemical, and biological processes superimposed upon a simulation of reservoir hydrodynamics. The latter are based upon mass-balance and limiting-nutrient concepts; they relate average eutrophication symptoms to external nutrient loadings, hydrology, and, morphometry using statistical models derived from a groups of lakes and/or reservoirs. Within their application ranges, empirical methods offer certain advantages in terms of simplicity and limited data requirements. Early empirical models developed by Dillon and Rigler(5), Vollenweider(6), and several others were based primarily upon data from northern lake data sets; their applicability to reservoirs is questionable because of lake/reservoir and regional differences in characteristics which influence nutrient dynamics, including morphometry, hydrology, and sedimentation (7).

As summarized by Reckhow and Chapra (8), empirical approaches generally involve the linkage of two types of models:

(1) Nutrient Balance Models relate pool or discharge nutrient levels to external nutrient loadings, morphometry, and hydrology.

(2) Eutrophication Response Models describe relationships among eutrophication indicators within the pool, including nutrient levels, chlorophyll-a, and transparency.

Generally, models of each type must be employed to relate external nutrient loadings to lake or reservoir water quality responses. In the absence of loading information, however, application of eutrophication response models alone can provide useful diagnostic information on existing water quality conditions and controlling factors.

Lake nutrient-balance models have generally evolved from a simplistic "black-box" representation which treats the impoundment as a continuous stirred-tank reactor at steady-state and the sedimentation of phosphorus as a first-order reaction (Figure 1). Using mass-balance data from groups of lakes, the sedimentation terms are empirically calibrated for predicting spatially- and temporally-averaged conditions. Model inputs can be expressed in three terms:

(1) Inflow Total Phosphorus Concentration (nutrient supply factor);

(2) Mean Depth (morphometric factor);

(3) Hydraulic Residence Time (hydrologic factor).

Response models (Figure 2) typically consist of bivariate regression equations relating each pair of response measurements (e.g., phosphorus/chlorophyll, chlorophyll/transparency, etc.). Phosphorus is assumed to control algal growth and other eutrophication-related water quality conditions.

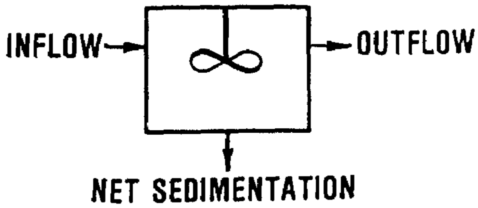

Figure 1. Lake Model Segmentation

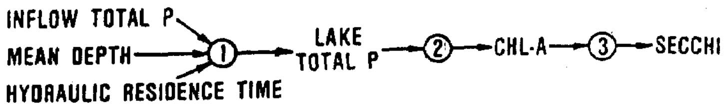

Figure 2. Lake Eutrophication Model Network

In adapting this modeling approach for use in reservoirs (2,3), modifications have been designed to account for the following:

(1) Effects of nonlinear sedimentation kinetics; a second order kinetic model appears to be more general than a first-order model both for predicting among-reservoir, spatially-averaged variations and for predicting within-reservoir, spatial variations in total phosphorus and total nitrogen;

(2) Effects of inflow nutrient partitioning (dissolved vs. particulate or organic vs. inorganic); because of differences in biological availability and sedimentation rates, reservoir responses appear to be much more sensitive the ortho-phosphorus loading component than to the non-ortho (total - ortho) component;

(3) Effects of seasonal variations in nutrient loadings, morphometry, and hydrology; pool water quality conditions are related more directly to seasonal than to annual nutrient balances in impoundments with relatively high flushing rates;

(4) Effects of algal growth limitation by phosphorus, nitrogen, light, and flushing rate; simple phosphorus/chlorophyll-a relationships are of limited use in reservoirs because nitrogen, light, and/or flushing rate may also regulate algal growth, depending depending upon site-specific conditions;

(5) Effects of spatial variations in nutrients and related variables; spatial variability in trophic state indicators is significant in many reservoirs (in some cases, spanning from oligotrophic to hypereutrophic conditions, Figure 3). and predictions of "average" conditions are of limited use.

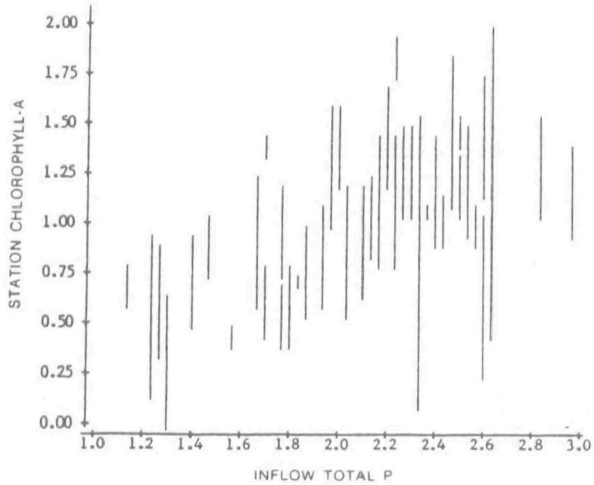

Figure 3. Ranges in Station-Mean Chlorophyll-a (PPB) vs. Reservoir Inflow Total Phosphorus Concentration (PPB), LOG10 Scales

To permit simulation of spatial variations, nutrient balance models are implemented in a spatially segmented framework which accounts for advection, dispersion, and sedimentation (Figure 4). While each segment is modeled as vertically mixed, the methodology is applicable to stratified systems because the sedimentation rate formulations have been empirically calibrated to data from a wide variety of reservoir types, including well-mixed and vertically stratified systems. The revised model network (Figure 5) is designed to improve generality (vs. Figure 2) by incorporating additional independent variables and controlling factors found to be important in model testing (2,3). When the network is applied to predict spatially-averaged conditions in 40 CE reservoirs, coefficients of determination (R-squared values) are 91% for Total P, 88% for Total N, 80% for Chlorophyll-a, 86% for Secchi Depth, and 92% for Hypolimnetic Oxygen Depletion Rate. Model structures and coefficients have been tested against several independent lake and reservoir data sets.

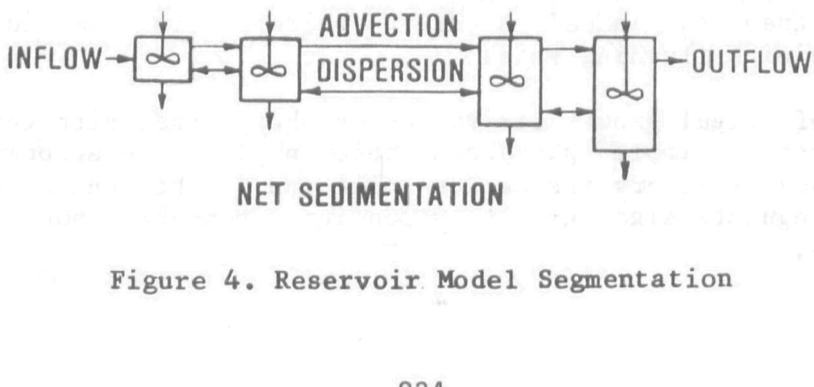

Figure 4. Reservoir Model Segmentation

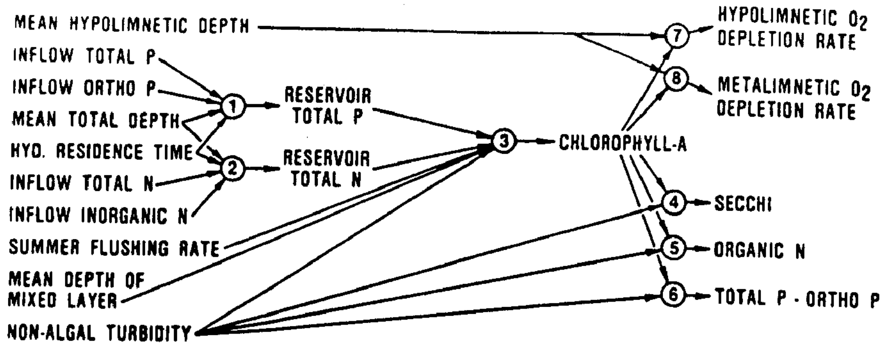

Figure 5. Reservoir Eutrophication Model Network

APPLICATIONS MANUAL AND SOFTWARE

Figure 6 depicts basic steps involved in applying the methodology, as described in the applications manual (4). Three computer programs have been written to assist at various stages of the analysis. The functions of these programs are outlined below:

(1) FLUX: estimation of tributary mass discharges (loadings) from concentration data and continuous flow records;

(2) PROFILE: display and reduction of pool water quality data;

(3) BATHTUB: implementation of nutrient-balance and eutrophication-response models in a spatially segmented hydraulic network.

Each program is written in FORTRAN-66, a language which is highly transportable among computer systems. The basic structure and functions of each program are outlined below.

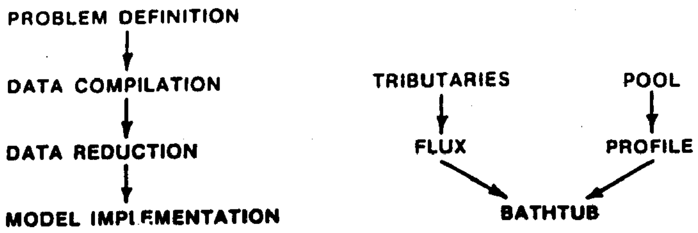

Figure 6. Assessment Pathways

FLUX is an interactive program for estimating loadings or mass discharges passing a tributary monitoring station over a given period (Figure 7). The loading estimates can be used in formulating reservoir nutrient balances over annual or seasonal averaging periods appropriate for application of empirical eutrophication models. The function of the program is to interpret water quality and flow information derived from intermittent grab or event sampling to estimate mean (or total) loading over the complete flow record between two dates.

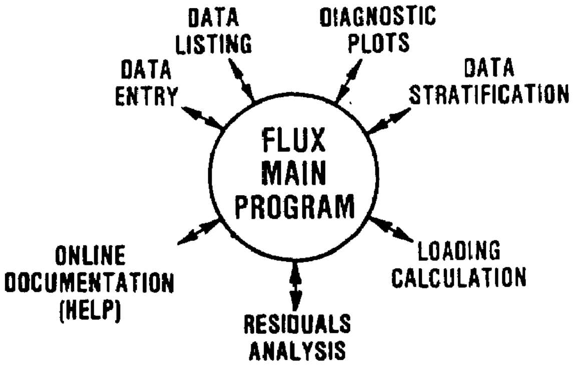

Figure 7. FLUX Schematic

Since the appropriate loading calculation method depends partially upon the concentration/flow/seasonal dynamics characteristic of a given station and component and upon the sampling program design, five alternative calculation methods are provided. An option to stratify the samples into groups based upon flow and/or date is also included. In many cases, stratifying the sample increases accuracy and reduces potential biases in loading estimates (9,10). The variances of the estimated mean loadings are calculated to provide relative indications of error. A variety of graphic and statistical diagnostics are included to assist the user in evaluating data adequacy and in selecting the most appropriate calculation method and stratification scheme for each loading estimate. The program can also be used to improve the efficiencies of monitoring programs designed to provide data for calculating loadings and reservoir mass balances by optimizing sampling effort among flow strata.

PROFILE is an interactive designed to assist in the analysis and reduction of pool water quality measurements (Figure 8). The user supplies a data file containing basic information on the morphometry of the reservoir, monitoring station locations, surface elevation record, and water quality monitoring data referenced by station, date, and depth.

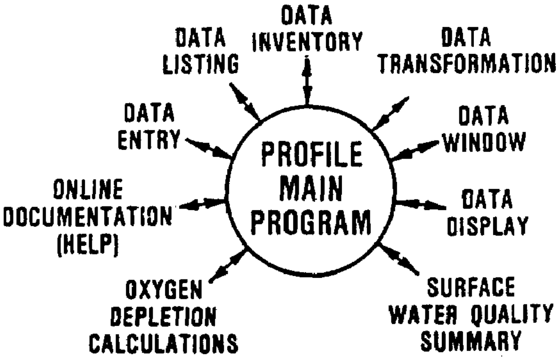

Figure 8. PROFILE Schematic

PROFILE's functions are in three general areas:

(1) display of concentrations as a function of elevation, location, and/or date, using a variety of one-, two- and three-dimensional formats;

(2) robust calculation of mixed-layer summary statistics and standard errors in a two-way table format (spatial x temporal);

(3) calculation of hypolimnetic and metalimnetic oxygen depletion rates from temperature and oxygen profiles;

Given adequate pool monitoring data from a particular reservoir, PROFILE assists the user in developing an appreciation for spatial and temporal variability in the reservoir. This may lead to refinements in monitoring program design. Summary statistics calculated in a uniform manner characterize reservoir trophic status and can be compared with predictions of nutrient-balance and eutrophication-response models in subsequent modeling steps.

BATHTUB facilitates application of empirical models to morphometrically complex reservoirs. The program performs water and nutrient balance calculations in a steady-state, spatially-segmented hydraulic network which accounts for advective transport, diffusive transport, and nutrient sedimentation (Figure 4). Eutrophication-related water quality conditions are predicted using empirical relationships tested for reservoir applications (Figure 5). As indicated in Figure 6, applications of BATHTUB normally follow use of FLUX for reducing tributary monitoring data and use of PROFILE for reducing pool monitoring data, although use of the data-reduction programs is optional if independent estimates of tributary loadings and/or average pool water quality conditions are used.

To reflect data limitations or other sources of uncertainty, key inputs to the model (flows, loadings, morphometry, observed pool concentrations, etc.) can be specified in probabalistic terms (mean and coefficient of variation). Outputs are expressed in terms of a mean value and coefficient of variation for each mass balance term and response variable. Output coefficients of variation are based upon a first-order error analysis (11) which accounts for input variable uncertainty and inherent model error (Figure 9).

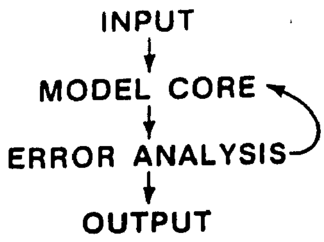

Figure 9. BATHTUB Schematic

Potential applications of BATHTUB can be broadly outlined in the following categories:

(1) Diagnostic:
formulation of water and nutrient balances, including identification and ranking of potential error sources;
ranking of trophic state indicators in relation to user-defined reservoir groups and/or the CE reservoir data base;
identification of factors controlling algal production;

(2) Predictive:
assessing impacts of changes in water and/or nutrient loadings;
assessing impacts of changes in mean pool level or morphometry;
predicting long-term-average conditions in a new reservoir;
estimating loadings consistent with water quality objectives.

BATHTUB operates in a batch model (non-interactive) and generates output in ten optional formats, as appropriate for specific applications. The water balances are expressed as a system of simultaneous linear equations which is solved via matrix inversion to estimate the advective outflow from each model segment. The mass balances are expressed as systems of simultaneous nonlinear equations which are solved iteratively via Newton's Method (12).

Through appropriate configuration of model segments, BATHTUB can be applied to a wide range of reservoir morphometries and management problems. Possible segmentation schemes include:

(A) Single Reservoir, Spatially Averaged
(B) Single Reservoir, Spatially Segmented
(C) Partial Reservoir or Embayment, Spatially Segmented
(D) Single Reservoir, Spatially Averaged, Multiple Load Scenario
(E) Collection of Reservoirs, Spatially Averaged
(F) Network of Reservoirs, Spatially Averaged

Segments can be modeled independently or linked in a one-dimensional, branched network. Multiple external sources and/or withdrawals can be specified for each segment. Within certain limitations, combinations of the above schemes are also possible.

Typical model output for Segmentation Schemes B-F is illustrated in Figure 10. Observed phosphorus concentrations in these plots are based upon mixed-layer, growing-season measurements in the reservoir or lake pool. Estimated concentrations are calculated from external phosphorus loadings using empirical second-order sedimentation models developed for CE reservoirs (3). The program generates similar plots for each response variable. Schemes A-C deal with single reservoirs or embayments. In Schemes D and E, model segments are run "in parallel" to permit simulation of alternative loading scenarios or independent reservoirs, respectively. Scheme E is particularly useful for obtaining regional perspectives on trophic status and load/response relationships in a collection of lakes or reservoirs. Scheme F permits routing of water and nutrients through a network of lakes or reservoirs, each of which may be spatially segmented.

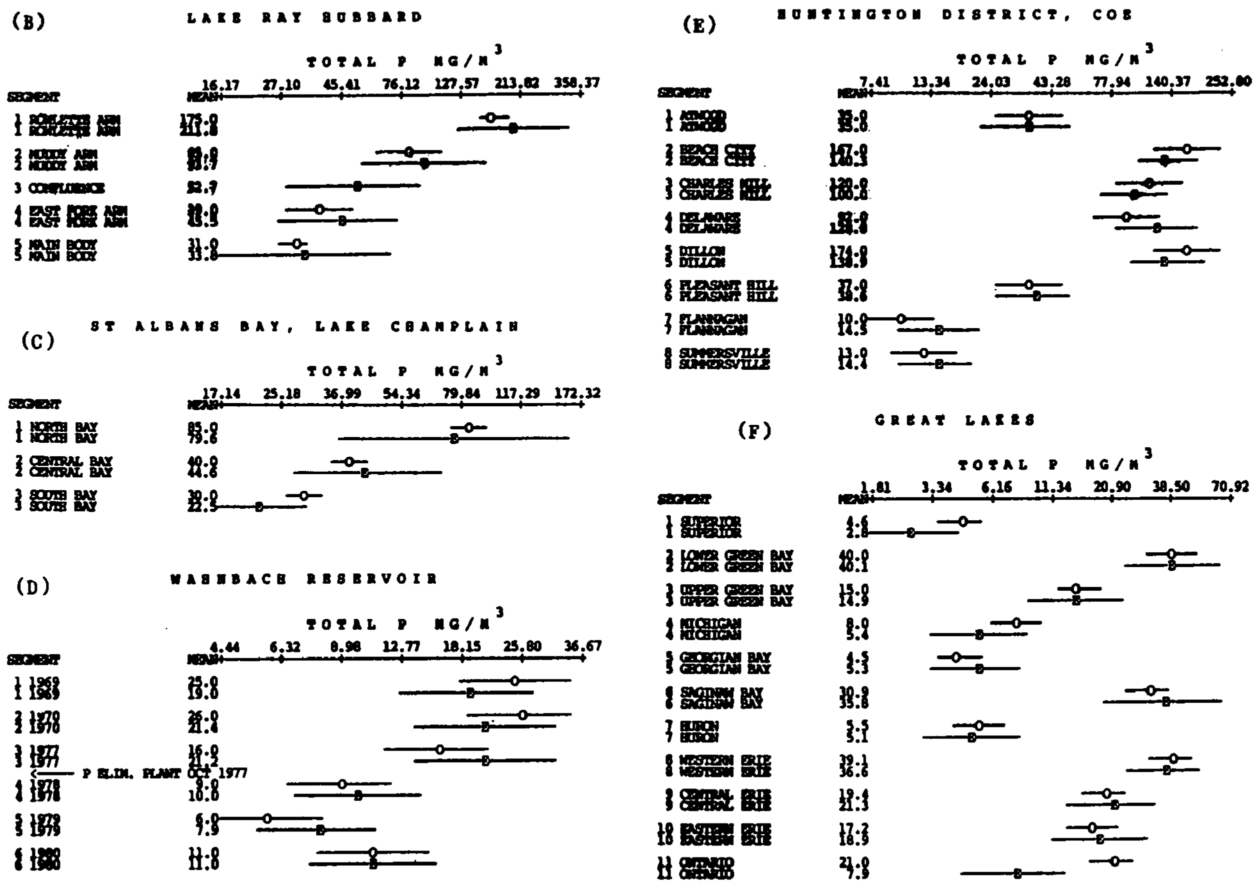

Figure 10. Observed (O) and Estimated (E) 90% Confidence Ranges for Mean Total Phosphorus Concentrations Illustrating Various Types of BATHTUB Segmentation Schemes

CONCLUSIONS

Results of a recent research project on empirical modeling techniques for reservoir eutrophication have been summarized. Detailed documentation of model origins, structures, and limitations can be found in the project reports (1,2,3). The applications manual and software (4) will be generally available in mid-1985.

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MODELING OF PHOSPHORUS CONCENTRATIONS FROM DIFFUSE SOURCES

by: D.J. Andrews
Marshall Macklin Monaghan Limited
Don Mills, Ontario
Canada M3B 2Y1

K.K.S. Bhatia
National Institute of Hydrology
Roorkee (U.P.)
India - 247 667

E.A. McBean
Department of Civil Engineering
University of Waterloo
Waterloo, Ontario
Canada N2L 3G1

ABSTRACT

The primary nutrients which control the trophic state of water bodies are nitrogen, phosphorus and carbon. Of these, phosphorus is recognised as the most limiting and most easily controlled because a primary source of phosphorus is frequently from domestic sewage. However, control of this particular nutrient involves high costs and even if the phosphorus input is controlled from point sources, a significant phosphorus presence may remain, as contributed by the uncontrolled diffuse sources. The focus of the paper is to link phosphorus concentration data with commonly-measured watershed characteristics such as daily flow. The data bases used in the study are, four watersheds of the Grand River (for a period of three years and nine months). The regression models for the most part, were based on flow or parameters which can be derived from flow. The dependent variables include a) total phosphorus, b) filtered reactive phosphorus concentration and c) total dissolved phosphorus concentration. The regression models were further based on seasonality. Useful regression models were produced on all the watersheds studied for the total phosphorus form of species. However, for other species the results were not very conclusive. The models had a tendency to overpredict low to median concentrations and to underpredict high concentrations. Conclusive results were obtained on the use of seasonal models.

INTRODUCTION

In recent years a lot of attention has been directed toward the eutrophication of water bodies (1). Such studies have led to the identification of primary nutrients - nitrogen, phosphorus and carbon in controlling the trophic state of lakes (2). Phosphorus is normally present in most natural waters in relatively low concentrations. Because it is essential to the plant growth, it has become the focus of attention in the entire issue of eutrophication which has led to a trend in wastewater treatment facilities to control output of this nutrient, often at very high costs. The possibility exists, however, that even with complete elimination of point source phosphorus, there will be sufficient phosphorus contributed through uncontrolled, diffuse sources to provide an adequate nutrient basis for the eutrophication process.

A primary difficulty in characterizing the diffuse sources is caused by their váriability, both spatially and temporally. Efforts aimed at characterizing the phosphorus contribution have therefore taken the form of agricultural plot studies or flux studies (3),(4). The agricultural plot studies, while able to give typical concentrations found in runoff from the test sites, do not account for the transport phenomena and therefore cannot specify the ultimate magnitude and time of delivery to the major water body. The flux studies, though overcoming this difficulty, are normally watershed-specific. Ideally, to properly estimate diffuse source loading, some form of model is required which links phosphorus concentrations with a commonly measured watershed parameter such as daily flow.

The purpose of this paper is to describe the success in formulating regression models, based on data collected from intensively measured watersheds. The regression models for various forms of phosphorus were formulated using flow (or variables derived from flow), various seasons and time. The models were developed for the Grand River Basin. In a related paper (5), these models are used in the St. John River Basin.

DATA USED IN THE STUDY

The data used for developing the regression models were for the Grand River (Figure 1). The data were provided by the Ontario Ministry of the Environment, covering the period from March 1974 to December 1977, for some 40 stations (6). Table 1 gives the major features of the various sampling sites (four in number).

METHODOLOGY

After a critical review of the models available for phosphorus modelling (7),(8),(9), it was observed that

a. Different models are required for different species of phosphorus (even within the same watershed) because of relative effects of each species on the environment and also because of runoff-dilution effects;

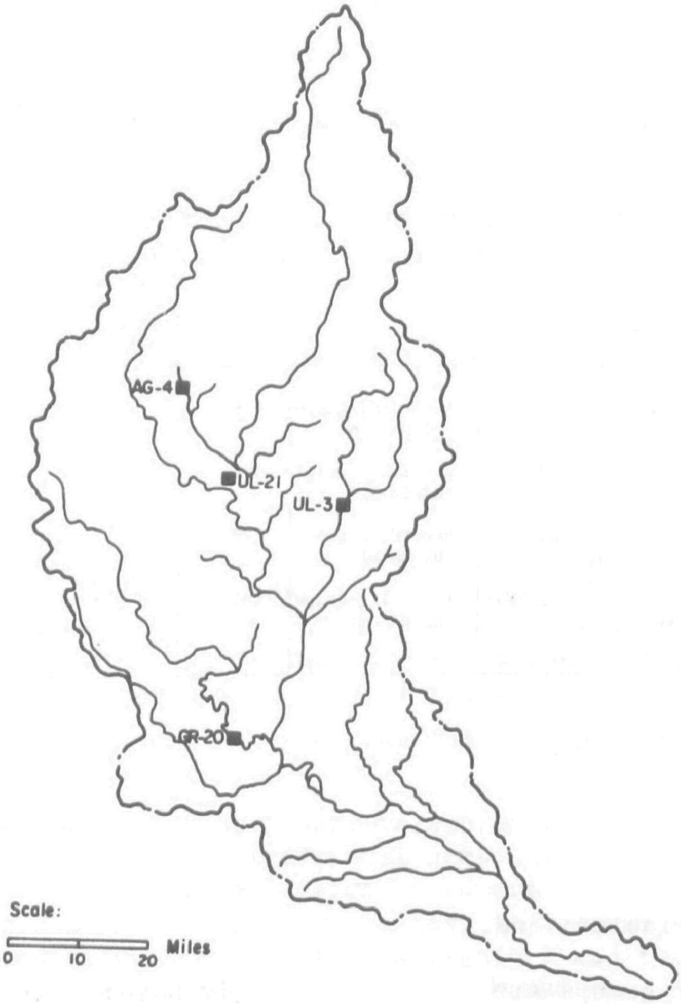

Figure 1. Sampling stations - Grand River

b. It is useful to include various phenomena in models such as seasonal variation, effect of antecedent conditions, and, physical effects of rainfall-runoff intensity;

c. Where possible, the effect of point sources should be minimized; and,

d. A primary aim should be towards the formulation of models that will predict the shape of the phosphorus profile.

VARIABLE DELINEATION

The only variable which is measured frequently, and which may be readily simulated in most watersheds, is flow. Consequently, this study has focussed primarily on parameters which may be derived from the flow records. Four major groups of variables have been derived from the flow records. They include:

TABLE 1. DETAILS OF VARIOUS GAUGING SITES

Site No.	Name of Site	Code No.	Location	Drainage Area (Miles^2)	Length of Data Set (No. of Samples) After Red.	Length of Data Set (Period)	Average Flow (cfs)
1	Canagagigue Creek -near Floradale	AG-4	Recording gauge 02 GA 036	6.9	256	Feb.19,1975 -Dec.13,1977	8.5
2	Nith River -near Canning	GR-20	Recording gauge 02 GA 010	398	183	Feb.25,1975 -Dec.19,1977	447
3	Speed River -near Guelph	UL-3	Recording gauge 02 GA 015	229	217	Feb.4,1975 -Dec.15,1977	213
4	Grand River -near West Montrose	UL-21	Recording gauge 02 GA 034	451	232	Feb.19,1975 -Dec.23,1977	483

1. Flow;
2. Antecedent flow;
3. State variables; and,
4. Time.

Each of these four groups are described briefly here:

1. Flow: The instantaneous flow at the time of measurement(Q_i) was included in order to determine the direct dependence between flow and concentration. The square of variable Q_i ($Q_i SQ$) and natural logarithm of Q_i(LNQ_i) were included to detect possible curvature which might exist in this relationship. The possibility of rising phosphorus concentrations lag the rising flow levels was tested using Q1 (flow lagged by one day) and the effect of normalizing the flow variable was considered using a variable VARQ defined as

$$VARQ = (Q_i - \bar{Q}) \div \sum_{i=1}^{n} (Q_i - \bar{Q})^2/n$$

where n is sample size and $\bar{Q}$ is the average flow over the n samples.

2. Antecedent Flow: Under this category, the following three variables were included to fulfill the above-given functions:

$$AQ = \text{Antecedent flow} = \sum_{j=1}^{j=3} (ADF)_j/3 \text{ where}$$

$(ADF)_j$ is the average daily flow recorded j days prior to the measurement.

AQSQ = square of variable AQ and

LNAQ = natural logarithm of the variable AQ.

3. State Variables: These variables were included to take into consideration certain aspects of flow regime not readily quantifiable in numeric terms. The state variable group consisted of two dummy variables and a single numeric value as given below:

   D1 - dummy variable, normally given a value of zero, but set to one during periods of rapidly rising flow (D1 = 1 if $Q_{i+1} \geq 1.15\ Q_i$)

   D2 - dummy variable, normally given a value of zero but set to one during periods of rapidly declining flow.

   QAQ - a numeric value defined as quotient of the variable Q and AQ (Q/AQ).

   The dummy variables are included so that some differentiation may be made between similar magnitudes of flow, when recorded on opposite limbs of the same hydrograph. The variable QAQ (not strictly a state variable) is included to consider the sharpness of the rise and fall of the flow regime at specific points of time.

4. Time: This was included to give some indication of the temporal effects on phosphorus concentration.

   No effort was made to create dummy variables to indicate season as there was sufficient data to run individual seasonal regressions. Variables were created, however, in an effort to ascertain the most important portion of each season. These variables, designated as DAY1, DAY2 and DAY3 are represented schematically in Figure 2 and represent various linear combinations of the days within each season.

   In addition, two temporal variables were defined which were not bounded by seasons, these being

   T - time in days from the last recorded major peak flow; and,

   DAY - consecutive day number set to zero for January 1, 1975.

   The variable T was included in order to determine the degree of dependence existing between phosphorus concentration and the accumulation period. This concept is similar to one used in various other simulation models (SWMM, STORM). The variable DAY was included to consider for any trend in the level of concentration over the full study period.

   The sixteen variables defined above comprised the complete set of independent variables employed.

5. Dependent variables include:

   TP: total phosphorus concentration which includes orthophosphates, condensed phosphates and originally bounded phosphorus;

   FRP: filtered reactive phosphate (phosphorus which passes through 1-2 μm filters); and,

   TDP: total dissolved phosphorus (phosphorus which passes through a 0.45 μm filter).

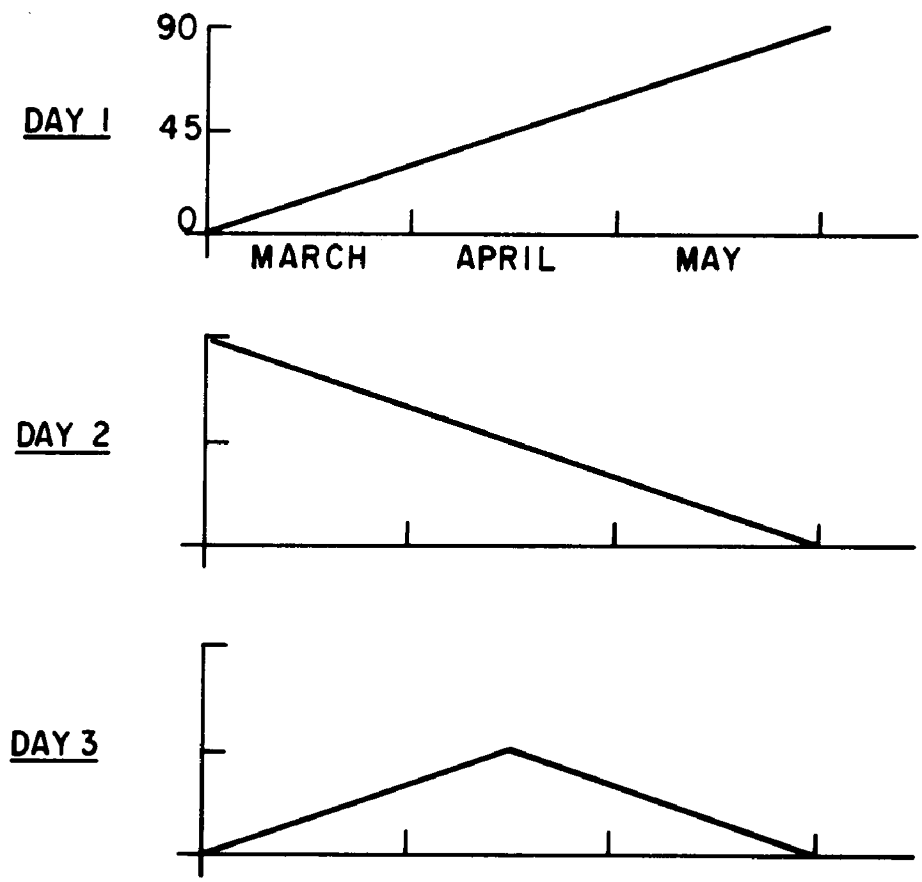

Figure 2. Definition of variables Day1, Day2, Day3.

Also utilized were the natural logarithms of TP, FRP and TDP. The natural logarithm was represented by LN. The units of concentrations were mg/ℓ and flow was measured in cubic foot per second.

SEASONAL BREAKDOWN

Regression analyses were performed in each case on the full data set. Six seasonal subsets as listed below were used:

1. R - Overall regression on data.
2. NSR - Non spring delineation of data, observations from March, April, and May of each year deleted.
3. WR - Winter regression, only data from December, January, and February included.
4. SR - Spring regression, only data from March, April, May included.
5. SUR - Summer regression, only data from June, July and August included.
6. FR - Fall regression, only data from September, October, and November included.

Numerals appended to the regression type indicate the sequence of the regressions (i.e. WR4 would designate the fourth regression completed, using the winter data set).

REGRESSION ANALYSIS

Regression analyses were performed using a computerized format developed for this study. The main control program, 'REGMASTER' utilized a number of sub-programs created to provide an additional data manipulation to ensure proper input to library regression routines (10). A flow diagram showing the procedure used in REGMASTER is shown in Figure 3.

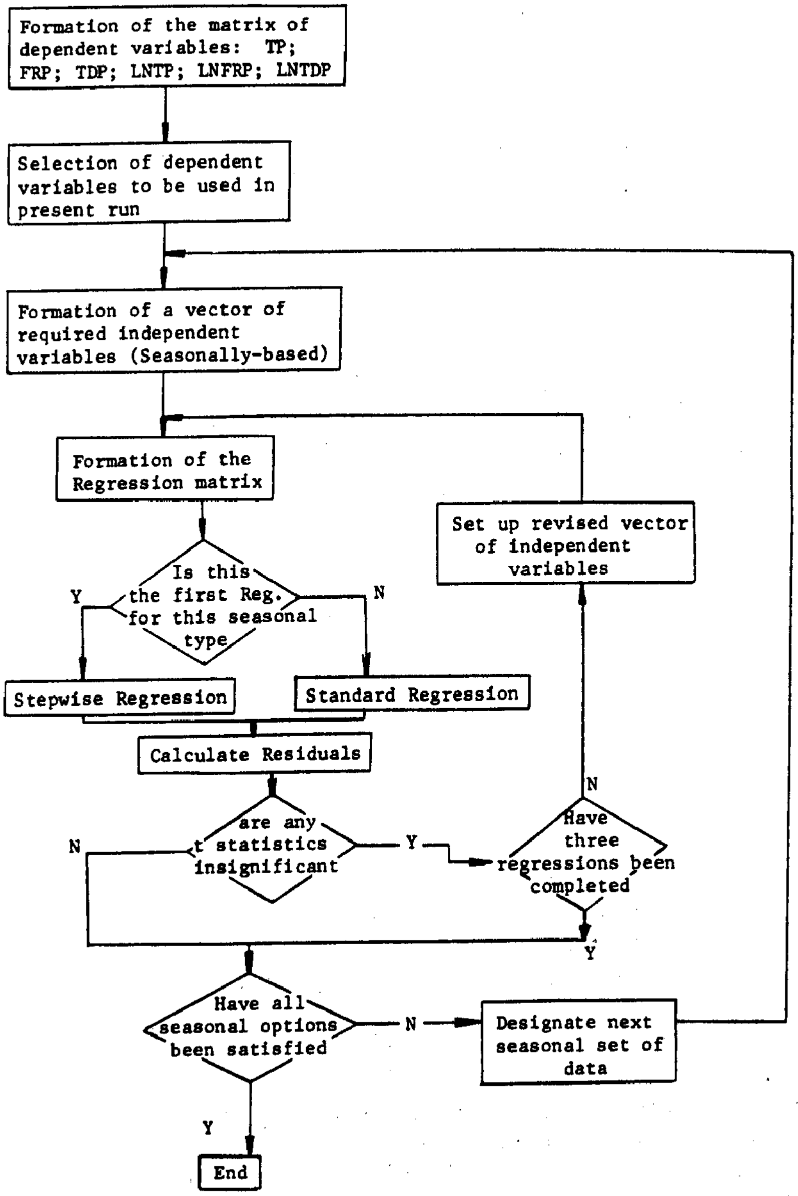

Figure 3. Flow diagram for REGMASTER

Preliminary regression analysis on all watersheds showed a consistent lack of significance in a number of variables outlined earlier. For this reason, all time variables, the flow variables Q1 and VARQ, and the dummy variable D2 were excluded from the standardized regression analysis. Regressions were run for each seasonal subset in each watershed using the remaining independent variables (Q, QSQ, LNQ, AQ, AQSQ, LNAQ, D1, QAQ) and the dependent variables (TD, FRP, TDP). Where some of these variables proved to be insignificant, they were eliminated and the regression was re-run. Insignificance of a variable was defined by a 't' statistic less than $t_{(0.995,n)}$ and a contribution to the total R^2 of less than 0.015. In some cases, important combinations of variables were missed by the computerized regression format, which was limited to a maximum of three runs in any particular season. Hence, careful attention was paid to the results of the computerized regression and auxiliary regressions were performed whenever a promising group of variables appeared to have been omitted. Following completion of the regression analysis, plots of predictions and residuals were completed. Visual analyses were performed to end the analysis (5),(10).

RESULTS AND DISCUSSIONS

1. Regressions for the Site AG-4

Six regressions (one for each seasonal breakdown) were selected from completed regressions. The details of pertinent statistics and various alternatives studied (like outliers, etc.) are discussed elsewhere (5). The selected regressions (based on best statistics) for total phosphorus, filtered reactive phosphorus and total dissolved phosphorus are given in Tables 2(a), 2(b) and 2(c) respectively. Table 2(b) and 2(c) also give the statistics.

TABLE 2(a). SELECTED REGRESSIONS -TP

REGRESSION TYPE	EQUATION
Overall (R2)	$Y = 7.30\times10^{-2} + 3.21\times10^{-3}\ AQ - 3.51\times10^{-2} LNAQ - 1.22\times10^{-5} AQSQ$ (t = 4.8) t = -7.2) (t = -3.5) $+ 7.37\times10^{-2}\ D1 + 2.24 \times 10^{-3} Q$ (t = 4.7) (t = 12.6)
Non Spring (NSR3)	$Y = .0825 + .0075\ AQ$ (t = 7.3)
Winter (WR5)	$Y = .0816 + .0030\ Q - .0177\ LNAQ$ (t = 6.8) (t = 3.6)
Spring (SR4)	$Y = .04105 + .00245\ Q + .07140\ D1$ (t = 14.2) (t = 2.8)
Summer (SUR4)	$Y = .10717 + .02389\ AQSQ + .01231\ QAQ - .08234\ AQ$ (t = 47) (t = 2.8) (t = -3.9)
Fall (FR6)	$Y = .10097 + .0072Q - .0547\ LNAQ$ (t = 4.6) (t = 4.6)

TABLE 2(b). SELECTED REGRESSIONS - FRP

Regression Type	Equation	SSR	SSU	+/- Ratio	F	R^2
Overall (R4)	Y= .02401 + .000537Q + .03781D1 (t = 7.3) (t = 4.4)	.181	.445	.53	51.5	.289
Non Spring (NSR3)	Y= .03418 + .02084LNQ - .01674 LNAQ (t = 6.5) (t = 5.1)	.048	.148	.75	21.2	.243
Winter (WR5)	Y= .04016-.01310LNAQ+.00256Q-2.01x10^{-5}QSQ (t= -4.5) t=4.7) (t = 3.3)	.013	.026	.43	10.5	.327
Spring (SR5)	Y= .01673 + .00056Q + .04734 D1 (t= 6.1) (t= 3.5)	.167	.261	.41	37.7	.390
Summer (SUR2)	Y= .0032 - .00761 LNAQ + .00347 AQSQ (t= -4.0) (t = 6.3)	.0048	.0031	.75	19.6	.611
Fall (FR3)	Y= .04133 + .02848LNQ - .02822 LNAQ (t= 3.6) (t = -3.5)	.038	.090	.52	7.4	.298

TABLE 2(c). SELECTED REGRESSIONS - TDP

Regression Type	Equation	SSR	SSU	+/- Ratio	F	R^2
Overall (R5)	Y= .04016 + .000511Q + .05075 D1 (t= 5.7) (t= 4.8)	.213	.633	.52	40.6	.24
Non Spring (NSR2)	Y= .05663 - .02538 LNAQ + .02694 LNQ (t= -5.6) (t = 6.1)	.081	.284	.52	18.8	.22
Winter (WR5)	Y= .06484 + .00146Q - .01319 LNAQ (t= 3.7) (t= -2.9)	.025	.113	.50	7.2	.179
Spring (SR2)	Y= .00209 + .00126 Q - 3.26X10^{-6}QSQ (t= 5.0) (t = -3.0) + .00450 D1 (t= 3.0)	.219	.291	.39	29.3	.42
Summer (SUR5)	Y=-.01915 + .03105AQ - .02831LNAQ (t= 5.3) (t = -5.7)	.015	.011	.40	16.7	.57
Fall	Y= .05848 - .03582LNAQ + .03347LNQ (t= -3.6) (t = 3.5)	.057	.132	.52	7.5	.300

In general, the regressions produced for AG-4 predict the measured concentrations adequately. Peaks and valleys associated with extremes in concentration are predicted well, especially during periods of high flow. During the periods of near constant low concentrations, the shape of the measured profile is, in general, well simulated by the regressions, but the magnitude of the prediction usually exceeds that measured. As the lower concentrations are the most frequent, there is therefore, a tendency to overpredict on a day-to-day basis. It should be noted that no lag effect was noticeable on a consistent basis when comparing simulated profiles to the measured profiles. It is felt that absence of this may be due to low magnitude of flow associated with this watershed.

While the regressions created for the FRP and TDP forms were in all cases significant, the statistics associated with each equation were not impressive. To compare the developed models with models consisting of sample mean, the models were constructed for each species based on the sample mean and a 99% confidence interval. This hypothesis concluded that the models developed earlier were much superior to the mean type of models.

2. Regressions for the Site GR-20

The regression analyses for the Nith River (GR-20) followed a format similar to that used in the study of the Canagagigue Creek watershed (AG-4). Those regressions which proved to be the best of each seasonal type are shown in Tables 3(a), 3(b) and 3(c) for the three types of species of phosphorus. The detailed statistics are given elsewhere (5).

TABLE 3(a). SELECTED REGRESSIONS - TP

REGRESSION TYPE	EQUATION
Overall (R2)	$Y = .0327 + .000105Q$ $(t = 27.4)$
Non Spring (NSR3)	$Y = .00105 + .00021Q - 3.49\text{x}10^{-8}QSQ$ $(t=12.1)$ $(t= -7.2)$
Winter (WR2)	$Y = -.32394 + .07347\ LNQ$ $(t=15.2)$
Spring (SR2)	$Y = .0330 + .15996\ D1 \times 8.773\text{x}10^{-5}Q$ $(t=3.0)$ $(t=10.2)$
Summer (SUR3)	$Y = .02991 + 1.036 \times 10^{-6}QSQ$ $(t = 6.0)$
Fall (FR4)	$Y = .00536 + .00023\ Q - .06535\ D1$ $(t=14.3)$ $(t= -4.4)$

TABLE 3(b). SELECTED REGRESSIONS - FRP

REGRESSION TYPE	EQUATION
Overall (R2)	$Y = -.06606 - 2.3875 \times 10^{-9}\ QSQ + 2.7296 \times 10^{-5}\ Q + 1.3989 \times 10^{-2} LNQ$ (t = 05.3) (t = 4.7) (t = 3.7)
Non Spring (NSR2)	$Y = -.084 - 3.48 \times 10^{-3}\ QAQ - 1.23 \times 10^{-2}\ D1 + 1.81 \times 10^{-2}\ LNQ$ (t = -5.1) (t = -2.7) (t = 5.3) $+ 4.06 \times 10^{-5}\ Q$ (t = 8.0)
Winter (WR2)	$Y = -.0912 + 2.09 \times 10^{-2} LNQ - 3.91 \times 10^{-3}\ QAQ + 3.39 \times 10^{-5}\ Q$ (t = 4.3) (t = -6.0) (t = 6.4)
Spring (SR6)	$Y = -.12537 + .02632\ LNQ$ (t = 9.3)
*Summer (SUR4)	$Y = -.1654 - .0153 D1 - .0084\ QAQ + .0376\ LNQ$ (t = -1.8) (t= -1.32) (t = 2.62)
Fall (FR5)	$Y = -.01801 - 3.469 \times 10^{-2}\ D1 + 1.467 \times 10^{-4} Q - 4.572 \times 10^{-8} QSQ$ (t= -5.6) (t = 7.2) (t = -3.2)

*Regression is not significant, nor are any of the independent variables. Removal of any variable results in a reduction of the significance of the other variables.

TABLE 3(c). SELECTED REGRESSIONS - TDP

REGRESSION TYPE	EQUATION
Overall (R2)	$Y = -.16327 + 3.5757 \times 10^{-2} LNQ - 6.6067 \times 10^{-10}\ QSQ$ (t = 14.7) (t = -3.8)
Non Spring(NSR2)	$Y = -.2004 - 3.563 \times 10^{-3} QAQ + 4.424 \times 10^{-2}\ LNQ$ (t = -3.8) (t = 11.5) $- 1.834 \times 10^{-2}\ D1 + 7.198 \times 10^{-9}\ QSQ$ (t = -2.9) (t = 5.3)
Winter (WR3)	$Y = -.2898 + .0636\ LNQ - .0051\ QAQ$ (t = 8.2) (t = -3.1)
Spring (SR3)	$Y = -.1065 + .0251\ LNQ$ (t = 7.0)
Summer (SUR4)	$Y = .00328 + 5.0399 \times 10^{-7}\ QSQ$ (t = 7.8)
Fall (FR2)	$Y = .4020 + 6.904 \times 10^{-4} Q - 4.19 \times 10^{-2} D1 - 2.36 \times 10^{-7}\ QSQ$ (t = 6.9) (t = -5.7) (t = -5.7) $- 2.69 \times 10^{-2}\ QAQ - 7.95 \times 10^{-2} LNQ - 1.35 \times 10^{-2}\ LNAQ$ (t = -4.5) (t = -4.1) (t = -3.4)

The statistics showed very encouraging results with the exception of the summer period as all values of F exceeded 100 and R^2 values exceeded 0.7. The seasonally aggregated model produced the best residual statistics. The statistics achieved for the filtered reactive phosphorus were, in general, inferior to those calculated for the total phosphorus form. Still the model R2 is the best suited for this form of species. The results achieved for total dissolved phosphorus were found to be better than FRP species.

3. Regressions for the Site UL-3

Significant regressions were obtained for all model types with the exception of the fall model, in the case of total phosphorus. Table 4 gives the 'best' regressions for the TP species.

TABLE 4. SELECTED REGRESSIONS - TP

REGRESSION TYPE	EQUATION
Overall (R2)	$Y = -.2661 + 2.756 \times 10^{-7} QSQ + .2436\ QAQ - 6.348 \times 10^{-4}\ Q$ (t = 18.6) (t = 13.5) (t = -11.3)
	$+ 5.333 \times 10^{-4}\ AQ - 2.028 \times 10^{-7}\ AQSQ + .1540\ LNAQ - .1415\ LNQ$ (t = 6.0) (t = -5.4) (5 = 5.3) (t = 4.9)
Non Spring (NSR2)	$Y = -.1265 + .1334\ QAQ + .1200\ LNAQ - .1162\ LNQ$ (t = 10.5) (t = 7.0) (t = 6.8)
Winter (WR4)	$Y = -.0467 + .0625\ QAQ + 9.6 \times 10^{-5}\ AQ$ (t = 7.6) (t = 4.4)
Spring (SR3)	$Y = -.1785 + 3.378 \times 10^{-7}\ QSQ - 9.146 \times 10^{-4}\ Q + .2198\ QAQ$ (t = 17.8) (t = -11.6) (t = 12.4)
	$+ 8.552 \times 10^{-4}\ QSQ - 2.840 \times 10^{-7}\ AQSQ$ (t = 8.5) (t = -6.7)
Summer (SUR4)	$Y = .0274 + 8.753 \times 10^{-7}\ QSQ - 6.510 \times 10^{-7}\ AQSQ$ (t = 12.2) (t = -3.4)
Fall (FR3)	$Y = .00658 + .01618\ QAQ$ (t = 1.9)**

**Regression is not significant, nor is the independent variable

The statistics achieved, varied greatly from one regression to the next. In general, the statistics appeared to be excellent for the overall and the spring regressions. The winter, non-spring and summer regressions showed only adequate statistics by comparison. In the cases of FRP and TDP, due to extreme skew in the data, it is felt that use of the overall regression analysis proved to be futile (5), and regression would produce misleading results. It is recommended, therefore, that where necessary, prediction of FRP and TDP within this watershed should be accomplished through the use of their respective mean concentrations.

4. Regressions for the Site UL-21

It is noteworthy that this station was the only one selected on the main stem of the Grand River. While it drains predominantly agricultural land, it lies downstream of the sewage treatment plant at Fergus and Elora. In addition, the Grand River is controlled above the sampling site at the Shand Dam. This site was chosen in order to ascertain difficulties which might arise in areas not dominated by agricultural landuse.

Significant regressions were achieved for all seasonal types with the exception of winter data subset. Table 5 gives the best regression models for TP. In general, for other forms of phosphorous, the regressions produced in this watershed were fairly weak and highly oriented towards relatively small number of major events, which may be due to the controlling Shand Dam. Efforts were made to discern the effect of reservoir by using differencing techniques on data collected at sampling sites, just below the Shand Dam. Unfortunately, no firm conclusions were possible due to inconsistency in the sampling dates between the two sites. It is felt that the 'best' possible model available would be based simply on a series of seasonal means. In general, it is felt that any attempt to predict these species within this watershed, based upon information presently available, would lead to misleading and inaccurate results.

TABLE 5. SELECTED REGRESSIONS - TP

REGRESSION TYPE	EQUATION
Overall (R4)	$Y = .44167 + 2.303 \times 10^{-4}\,Q + 5.865 \times 10^{2}\,QAQ - 9.045 \times 10^{-2}\,LNQ - 1.944 \times 10^{-8}\,QSQ - 7.623 \times 10^{-9}\,AQSQ$ (t = 5.3) (t = 6.3) (t = -4.7) (t = -4.1) (t = -3.4)
Non Spring (NSR2)	$Y = .1484 - .1965\,LNQ + .1008\,QAQ + .1619\,LNAQ + .0817\,D1 + 3.465 \times 10^{-8}\,QSQ$ (t = -3.6) (t = 4.4) (t = 3.0) (t = 2.9) (t = 7.7)
Winter** (WR4)	$Y = .5369 + .00027\,Q - .1134\,LNQ + .0136\,LNAQ$ (t = 1.3) (t = -.7) (t = .1)
Spring (SR2)	$Y = .0027 + .0064\,QAQ + 2.187 \times 10^{-5}\,Q$ (t = 7.6) (t = 4.0)
Summer (SUR3)	$Y = .3798 + .1718\,D1 - .0694\,LNQ + 2.45 \times 10^{-4}\,Q$ (t = 5.0) (t = -2.8) (t = 9.7)
Fall (FR3)	$Y = -1.589 + 7.008 \times 10^{-7}\,QSQ - 1.343 \times 10^{-3}\,Q + .3045\,LNAQ - 8.798 \times 10^{-8}\,AQSQ + .2595\,QAQ$ (t = 4.8) (t = -4.2) (t = 3.7) (t = -3.3) (t = 5.9)

**Regression Not Significant; No Variables Significant.

5. Comparison of Selected Regression Analyses

The models selected for the prediction of TP in each of the four Grand River watersheds are shown in Table 6. Associated statistics are shown in Table 7. No general form may be associated from the models selected, although the basic flow variable (Q) appears to be of predominant importance in most of the regressions. The state variable QAQ and the logarithmic transform of the basic antecedent flow variable (LNAQ) also appear frequently in the selected models. It does unquestionably show that the models developed cannot be transformed between watersheds within the Grand River system without some sort of normalization procedure (5).

TABLE 6. SELECTED MODELS - GRAND RIVER BASIN (TOTAL PHOSPHORUS)

Watershed	Applicable Season	Selected Models
AG-4	Spring	[TP] = .03618 + .0028 Q
	Summer	[TP] = .10717 + .0239 AQSQ + .0123 QAQ - .0823 AQ
	Fall	[TP] = .08905 + .00716 Q - .04971 LNAQ
	Winter	[TP] = .08164 + .00296 Q - .01766 LNAQ
*AG-4	All	[TP] = .08077 - .02073 LNAQ + .00317 Q
GR-20	All	[TP] = .00327 + .000105 Q
UL-3	All	[TP] =-.18378 + .17807 QAQ - .15947 LNQ + .16631 LNAQ
UL-21	All	[TP] =-.01747 + .07155 QAQ + 2.83 x 10^{-5} Q

*Not intially selected, included here due to General Form Prevalent in Other Watersheds
N.B. Variables are listed in order of significance

TABLE 7. MODEL STATISTICS - GRAND RIVER BASIN (TOTAL PHOSPHORUS)

Watershed	Season	N	SSR	SSU	R^2	F	+/- Ratio	Residual Statistics AMR	% of Mean	Max Res.	Min Res.
AG-4	Spring	118	2.02	0.43	.822	535	.63	.043	38	.284	-.127
	Summer	28	0.068	0.043	.610	12.5	1.00	.031	30	.110	-.055
	Fall	37	0.19	0.14	.575	23	.54	.049	46	.190	-.089
	Winter	69	0.095	0.134	.415	23	.60	.027	32	.271	-.049
*AG-4	All	252	2.31	0.86	.729	334	.60	.041	40	.263	-.144
GR-20	All	183	5.81	1.40	.806	752	.43	.043	39	.704	-.340
UL-3	All	166	0.19	0.11	.630	92	.59	.015	38	.133	-.087
UL-21	All	231	2.02	1.55	.565	149	.65	.042	46	.620	-.219

CONCLUSIONS

This work was initially aimed at extending the ability to predict phosphorus concentration for three different sub-species - Total Phosphorus (TP), Filtered Reactive Phosphorus (FRP), and Total Dissolved Phosphorus (TDP). Useful regression models were produced on all the Grand River watersheds studied, for the TP form. Attempts to form useful models for the FRP and TDP forms met with variable success however, and it must be concluded that, in general, the simulation of these species was not possible using the available techniques. All conclusions, therefore, are directed towards the TP specie.

In general, the models relied heavily on the standard flow variable (Q) and flow curvature forms (QSQ and LNQ). In seasonally-based regressions the variable QAQ and LNAQ proved useful. The presence of QAQ in the model indicates the importance of hydrograph shape while the presence of LNAQ indicates a relationship between concentration and antecedent conditions. All models showed a tendency to overpredict low to median concentrations (to a minor extent) and underpredict higher concentrations (to a greater extent). This may be due to skew, evident in most of the data sets and to the mathematical techniques used in multiple regression analysis. In most of the cases, seasonally-based models proved to be better than non-seasonal models. It is noteworthy that the predicted shape of the phosphorus profile corresponded well in all cases to the measured profile. The models therefore show a good potential with respect to predictive ability. Further study may show the identification of parameters which will help to reduce the error present in the prediction of extreme values.

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A MODEL FOR ASSESSING THE COST-EFFECTIVENESS OF AGRICULTURAL BMP IMPLEMENTATION PROGRAMS ON TWO FLORIDA BASINS[1]

by: C.D. Heatwole, Graduate Assistant
A.B. Bottcher, Associate Professor
L.B. Baldwin, Associate Professor
Agricultural Engineering Dept. - Univ. of Florida

ABSTRACT

A model was developed to evaluate the cost-effectiveness of alternative BMP implementation schemes on two agricultural basins in Florida. The model selectively applies the desired BMPs throughout the basin, estimates the associated costs, and predicts the water quality improvement (reductions in nitrogen and phosphorus). Fifteen BMP scenarios were evaluated to aid in prioritizing BMPs for implementation in these basins. Applying the maximum level of BMPs is estimated to cost around $1.2 million (annually), while the four most cost-effective BMPs would cost only one quarter as much, yet provide approximately 90% of the water quality improvement.

INTRODUCTION

The Lower Kissimmee River, LKR, basin (2010 sq km) and the Taylor Creek-Nubbin Slough, TCNS, basin (498 sq km) lie in the "flatwoods" region of Florida, an area characterized by very flat sandy soils and a high water table that fluctuates from the surface to 2 meters deep. Ranching and dairying are the primary land uses of these basins. In 1980 the Taylor Creek watershed was 69% improved pasture, 16% forest and range, 3% citrus, 4% urban, and 8% miscellaneous (2). The Nubbin Slough watershed has similar land use, while the larger Lower Kissimmee River basin is less developed (more unimproved pasture, forest and range instead of improved pasture).

These basins lie on the north side of Lake Okeechobee and contribute about 35% of the water flowing into the lake. However, they also discharge a

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disproportionate amount of nutrients into the lake, accounting for 49% of the phosphorus, and 31% of the nitrogen annually added to the lake waters (7). This 31% contribution of nitrogen, though lower than the water contribution, is still proportionally higher than loads coming from neighboring sandy watersheds north of the Lake. The organic soils south of the Lake. The organic soils south of the Lake contribute disproportionality high amounts of nitrogent making the number only appear low. Reduction of these nutrient loads is being tackled at the upland source areas through the use of appropriate best management practices (BMPs). A list of BMPs applicable to Florida conditions has been compiled (4), but quantifying the effects of BMPs is a much more difficult task.

Detailed hydrologic-water quality models are the primary means of evaluating BMP effectiveness (3). However, these models generally do not directly consider the economics of the different BMP alternatives. The goal of this project has been to develop a model for the LKR and TCNS basins whereby the cost-effectiveness of different BMP implementation scenarios can be evaluated on a basin scale.

MODELING APPROACH

The U.S. Army Corps of Engineers has developed an extensive geographic database of the LKR and TCNS basins. The two basins were divided (200 m by 250 m grid) into 50,165 five hectare (12.35 acre) cells. Data for each cell includes basin and sub-basin codes, soil group, hydrologic soil group (A/B/C/D), presence of a stream in the cell, elevation (for flood plain cells), and land use for the years 1980, 2010, and 2030. Of this, hydrologic soil group, land use, and stream location data were used in this study. This 'cell' was the basic land unit used by the model for applying BMPs and in water quality modeling.

The objective of the model is to provide a tool for evaluating: different BMPs, different levels of application of a given BMP (e.g. 1 cm impoundment versus 2 cm impoundment), and various combinations of BMPs. The model has two major components. The first section of the model takes the desired BMP scenario, applies the BMPs to each applicable cell in the basin, and summarizes the costs of the 'applied' BMPs. The second component is a water quality model that predicts the average annual nutrient loading of nitrogen (N) and phosphorus (P) from the basin. This is compared against a base-line nutrient loading (predicted by the model for the 'no BMP' condition) to give the expected water quality improvement in kilograms of N and P reduced per year due to the BMPs. Model outputs for each BMP scheme considered are: water quality improvement, BMP costs, and cost effectiveness in terms of improvement per dollar cost.

BMP SELECTION AND COSTS

BMPs for the LKR and TCNS basins must be oriented to: keeping livestock away from drainageways, dispersing wastes for soil assimilation and plant uptake, proper fertilization and water management, and impounding runoff for nutrient attenuation. Specific BMPs used in this study were: fencing streams and wetlands that border pasture, constructing runoff detention basins, and impounding runoff from dairy barn lots for application to pasture and crop land.

BMPs were assigned by considering the land use, cattle density (for pasture land), hydrologic soil group, and the distance of the cell from a stream or wetland. Cells neighboring a stream cell would be expected to deliver greater amounts of N and P to streams than those not bordering the stream. Thus BMP application levels were decreased with increasing distance of a cell from a stream or wetland. For fencing of pasture from wetlands, decreasing amounts of fencing were applied as the distance of the adjoining wetland from a stream cell increased.

For cells identified as 'dairy', a method for estimating cattle density was needed. The number of milking cows at each dairy was known, but additional breakdown of the 'dairy' land into pasture or hayland was not available. Dairy cells were divided into three groups of decreasing cattle density ('cow pasture', 'dry cow pasture', and 'hayland'), based on distance from the barn cell. Cattle density was then computed as a function of the assigned dairy land use group and the size of the milking herd.

The cost of installing the BMPs was obtained by multiplying unit costs by the total amount of each BMP applied. Unit costs were 1984 estimates obtained from contractors and the USDA-SCS. Annual maintenance costs were also estimated. These costs were amortized over the expected life of the BMP to obtain an 'annual' cost. This annual cost estimate was used in the cost-effectiveness calculations.

WATER QUALITY MODEL

The water quality model is a combination of two sub-models. The first sub-model is a modified version of the CREAMS model (CREAMS-WT) which predicts nutrient and water yield from each individual cell. These results are then passed to the second sub-model, BASIN (developed by the authors), which integrates the cell results over the entire basin. The CREAMS model, developed by USDA-ARS (10), predicts runoff, erosion, and nutrient yield (nitrogen and phosphorus) for field-sized areas. A description of the modifications made for the CREAMS-WT sub-model will follow. The BASIN sub-model takes the nutrient predictions for each cell and predicts nutrient delivery to the edge of the nearest stream and at selected sub-watershed outlets in the basins. This modeling approach utilized all the information available at the cell level and enabled prediction for the entire basin.

THE CELL SUB-MODEL: CREAMS-WT

Objectives in the design of CREAMS (Chemicals, Runoff and Erosion from Agricultural Management Systems) were that the model should: 1) simulate major physical processes that control water balance, erosion, sediment yield, and movement of plant nutrients and pesticides, 2) use physically based parameters that can reflect changes in management systems, 3) be computationally efficient--operate on a daily time step, and 4) be field scale, since this is the common base for BMP selection (6). Because the parameters of the model are physically based, they can be estimated from site visits, maps, county soil survey reports, and the CREAMS manual. An additional advantage of physically based parameters is that the need for calibration is minimized.

The CREAMS-WT version of CREAMS was developed by Heatwole et al. (9) for the South Florida flatwoods. CREAMS-WT has the added ability to follow the fluctuating water table which strongly influences the hydrologic processes of this area. This improves the model conceptually, and yields better estimates of annual runoff and evapotranspiration.

To assure that the simulated results represented a long term average, simulations were run using 20 years of actual weather data. The first two years of the simulation were used to assure stable initial conditions, with the final 18 years used to obtain an average annual N and P load for each cell.

The simulation of BMPs with CREAMS-WT involves changing the model parameters (such as curve number and soil properties) to reflect changes the new practice would cause. One of the most important BMPs in this study, fencing, is not reflected directly in any of the CREAMS-WT parameters. Fencing was modeled by distributing animal waste (which CREAMS-WT considers as a 'fertilizer' application) between the pasture and the wetland or stream which it borders. The fraction of waste distributed to each depends on the degree of fencing.

THE BASIN SUB-MODEL: BASIN

The functions of the BASIN sub-model are: 1) Model the effects of BMP's (detention and retention basins) that cannot be simulated in CREAMS-WT, 2) Provide background loading for forests, native range, and other non-agricultural land use, 3) Attenuate the nutrient loads from cells to compute edge-of-stream loadings, 4) Compute nutrient loads due to cattle in wetlands (CREAMS-WT does not handle wetlands), and then attenuate these loads based on flow distance through the wetland to a stream, and 5) Attenuate nutrient loads from edge of stream to sub-watershed outlets.

The most important parameters for this model are the attenuation factors needed for the functions mentioned above. These were obtained by summarizing the results of many nutrient studies in controlled marshes and natural wetlands in South Florida (5).

MODEL CALIBRATION

In the water quality model, CREAMS-WT and BASIN are both designed to use physically based parameters thus minimizing the need for calibration. Very little calibration was actually needed, as comparison of model output with available observed data showed good correspondence for the initial parameter values chosen. The CREAMS-WT hydrology section was the primary component of the water quality model that was calibrated. Parameters were adjusted so that predicted average annual evapotranspiration and runoff would match the average annual values determined from water balance studies (1,12). Other non-calibrated parameters for the CREAMS-WT and BASIN sub-models were estimated from the previously mentioned sources and from personal observations of the area.

MODEL VERIFICATION

Verification of the model output was limited by the amount of available data. Comparison was made at two levels: the cell predictions by CREAMS-WT, and on a larger scale, the predictions of the BASIN sub-model. On these two levels, both runoff and nutrient yield were compared with measured data. Since limited data were used for calibration of runoff prediction on the cell scale, additional verification could not be done for this aspect of the model. On a watershed scale, estimates of the long term average annual runoff of 26.9 to 30 cm (1,12) compared favorably with the the basin-wide 27.8 cm average predicted by this model.

Nutrient concentrations and loads predicted by CREAMS-WT were checked against data from three pasture sites (8,11), and the output of the BASIN sub-model was compared with data from the Taylor Creek watershed (2). In each case, the predicted concentrations and loads fall within the range of observed values. Considering the scope and goal of the model and the expected variability in the measured data, the model predictions were considered acceptable.

RESULTS AND DISCUSSION

Fifteen different BMP scenarios were evaluated and are listed in Table 1 along with their resulting cost-effectiveness. In addition to the scenarios in Table 1, a 'do nothing' scenario was also simulated as the 'base line' for comparative evaluation of the other BMP scenarios. Water quality improvement was defined as the reduction in the average annual nitrogen and phosphorus load. Cost-effectiveness was then computed by dividing nutrient load reduction by the annual cost of the BMP scenario, giving units of kilograms reduced per dollar per year.

A variety of individual BMPs and combinations of BMPs are included in Table 1 as applied to both the LKR and TCNS basins. Because of the interaction between BMPs, the results from individual BMP evaluations cannot be added to estimate the response of combinations of BMPs. To determine the response of a particular group of BMPs, that combination of BMPs must be evaluated independently.

Several interesting relationships can be observed in Table 1. First, as the level of BMP application increases, the cost-effectiveness decreases (scenarios 1-3 and 13-15). Second, fencing is one of the more cost-effective practices (and also turns out to be the largest contributor to reducing the total nutrient load from the basins). Third, pasture impoundment turned out to be the least cost-effective of the practices evaluated. Forth, cost-effectiveness values for the TCNS basin are generally higher than the LKR basin, due in part to the higher intensity of land use in the TCNS basin. The higher nutrient loads will generally result in a higher percentage reduction thus higher cost-effectiveness, even though the remaining nutrient load may still be higher.

Since the streams are the primary pathways of mass transport of nutrients, the proximity of a cell to a stream has a dramatic impact on the actual

Table 1. BMP scenarios and their cost-effectiveness for the Taylor Creek-Nubbin Slough (TCNS) and Lower Kissimmee River (LKR) basins.

		COST EFFECTIVENESS			
		LKR		TCNS	
Scenario	Description	Nitrogen	Phos.	Nitrogen	Phos.
		------------(kg/$)------------			
1	All BMPs applied - high rate	0.42	0.11	0.67	0.15
2	" - medium rate	0.54	0.14	0.75	0.17
3	" - low rate	1.10	0.26	1.10	0.25
4	Dairy pasture runoff impoundment	0.06	0.03	0.04	0.02
5	" barn lot impoundment	2.00	0.47	3.60	0.24
6	" pasture fencing	1.90	0.51	2.60	0.55
7	Beef pasture impoundment	0.01	0.02	0.02	0.02
8	All dairy BMPs (barn and pasture)	0.19	0.04	1.40	0.33
9	Fencing of all beef pastures	1.40	0.33	1.70	0.38
	Beef pasture fencing:				
10	" - Intensively managed pasture	6.90	1.70	6.90	1.50
11	" - Improved pasture	1.50	0.36	1.90	0.41
12	" - Semi-improved pasture	0.43	0.11	1.90	0.40
	Row crop and citrus impoundment:				
13	" - high rate	0.39	0.18	0.08	0.06
14	" - moderate rate	0.63	0.30	0.13	0.10
15	" - low rate	1.90	0.86	0.18	0.14

nutrient delivery to a stream and therefore its potential nutrient reduction. Thus high nutrient producing activities, such as dairy barns, will have much less impact on water quality if located in 'upland' areas away from streams. The apparent reason for the relative ineffectiveness of pasture impoundment (scenarios 4 and 7) is that they were not used to treat nutrient loads in the streams or wetlands. The difference between the basins in row crop and citrus runoff impoundment effectiveness is likely due to the location of those areas relative to streams.

Using the cost-effectiveness data, the individual BMPs can be ranked. Considering both basins together (not a simple average of Table 1 values because of the differing areas involved), the BMPs in order of decreasing effectiveness are:

1. Fence dairy cows from streams and wetlands.
2. Fence intensively managed beef pasture and dry cow pasture from streams and wetlands.
3. Retain runoff from dairy barn holding lots and distribute on low density pasture or crop land.
4. Fence beef cattle on improved unirrigated pasture from streams and wetlands.

5. Impound (detain for nutrient attenuation) runoff from dairy cow pastures.

6. Impound runoff from intensively managed beef pasture and dry cow pasture.

7. Impound runoff from citrus and row crop land.

8. Impound runoff from unirrigated improved beef pasture.

The first four BMPs, if fully implemented could potentially reduce loadings at the edge-of-stream by 60% for N and 60% for P in TCNS, and 35% for N and 40% for P in the LKR basin. Note that these reductions cannot be directly projected to the basin outlet. The actual percent reduction further downstream would be less because of the natural background nutrient concentrations. Implementing the maximum BMP application (scenario 1) in both basins would have an initial cost of $5.2 million, with an annual cost of $1.2 million. In comparison, the four most cost-effective BMPs provide about 90% of the maximum load reductions found in scenario 1, but at an annual cost of only $0.3 million and an initial cost of $1.3 million. The benefit of this type of analysis is apparent.

An interesting observation, and one important to an actual implementation program, is the decrease in cost-effectiveness with increasing levels of application of a particular BMP. Even though a BMP may be the most cost-effective at one level of analysis, it may not be desirable to apply only that BMP in greater and greater amounts to meet some water quality goal. Rather, a point may be reached where a different BMP will become more cost-effective than continued use of the original BMP.

The model's assignment of BMPs to a particular cell is not to be taken as a field guide for actual implementation of a BMP program. The model only indicates the criteria for BMP application and an estimate of the total amount of BMPs applied in the basin. This will, however, be a very helpful guide for field personnel who ultimately must determine the proper location of the BMPs.

The model described in this article has been demonstrated to be an effective tool for evaluating various BMP applications in the LKR and TCNS basins. For a given BMP implementation program, the relative water quality improvement can be determined, and the cost of installing and maintaining those BMPs estimated. The cost-effectiveness of this particular program can then be compared with other BMP options. However, a final decision on a BMP strategy must depend on more than just cost-effectiveness. A specified level of improvement in water quality may be required, and also, there will generally be a limit on the available funding. The model provides an estimate of these three factors and can thus aid in the selection of the most cost effective BMP implementation program for these basins, taking into consideration water quality goals and financial constraints.

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ATTENDEES

Name	Representing
L.B. Baldwin	University of Florida
Tom Barnwell	EPA
B.A. Benedict	University of Florida
K.K.S. Bhatia	Indian National Institute of Hydrology
Del Bottcher	University of Florida
Linfield Brown	Tufts University
Kenneth Campbell	University of Florida
J.C. Capece	University of Florida
M-S Cheng	MD-National Capital Parks & Planning Commission
Ivan B. Chow	Applied Technology & Management, Inc.
B.A. Christensen	University of Florida
Bob Crabtree	University of Birmingham, U.K.
L.J. Danek	University of Florida
Robert Dickinson	University of Florida
Forrest Dierberg	Florida Institute of Technology
Henry Dorzback	University of Central Flordia
Jean Dorzback	University of Central Florida
Wayne Downs	University of Florida
Ann Dudek	Hazen & Sawyer
Gary Elwell	HKM Associates
Efi Foufoula-Georgiou	University of Florida
W.E.Fulong, Jr.	University of Florida
Richard Bigney	Camp, Dresser and McKee, Inc.

Bernard L. Golding	Hilburn Engineering
Philip Gronstal	Dallas Water Utilities
James P. Heaney	University of Florida
John Hendrix	
Nancy Hicks	University of Florida
Wayne C. Huber	University of Florida
William James	McMaster University
Kazuaki Kawamoto	Tokyo Metropolitan Sewage Works
Frank Kerino	
Vlademir Krejci	Swiss Federal Institute of Water Resources
Howard Litwack	State of New Jersey, Dept. of Water Resources
W-S Lung	University of Virginia
Ed McBean	University of Waterloo
Sharon Oakes	City of Gainesville
W.N. Pandorf	Environmental Science and Engineering
Curt Pollman	Environmental Science and Engineering
R.L. Powell	MD-National Capital Parks & Planning Commission
Leila Rhue	CH2M-Hill, Inc.
Mark Robinson	McMaster University
Ashok Shahane	FL. Dept. of Agriculture
Ed Sharp	City of Winnipeg
Charles Swallows	Geoscience
M.P. Timpe	Water & Air Research

William Tucker	Environmental Sciences and Engineering
S. Udhiri	MD-National Capital Parks & Planning Commission
A. Visvanatha	Underwood-McLellan Ltd.
William Walker, Jr.	Environmental Consultant
Marty Wanielista	University of Central Florida
Ray Wiles	Geoscience
John T. Westgate	Seaburn & Robertson Engineering
Ronald Wycoff	CH2M-Hill, Inc.
Yousef A. Yousef	University of Central Florida

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