SIMULATION OF THE EFFECTS OF PROPOSED CONSTRUCTION OF TWELFTH STREET EXTENSION AND OF FLOOD-PLAIN REFORESTATION ON FLOOD ELEVATIONS, CONGAREE RIVER NEAR COLUMBIA, SOUTH CAROLINA

By R. Erik Schuck-Kolben and Stephen T. Benedict

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 90-4056

Prepared in cooperation with the

SOUTH CAROLINA DEPARTMENT OF HIGHWAYS

AND PUBLIC TRANSPORTATION

Columbia, South Carolina

1992

U.S. DEPARTMENT OF THE INTERIOR

MANUEL LUJAN, JR., Secretary

U.S. GEOLOGICAL SURVEY

Dallas L. Peck, Director

For additional information write to:

District Chief
U.S. Geological Survey
Stephenson Center-Suite 129
720 Gracern Road
Columbia, South Carolina 29201

Copies of this report can be purchased from:

U.S. Geological Survey
Books and Open-File Reports Section
Federal Center
Box 25425
Denver, Colorado 80225-0425

CONTENTS

ILLUSTRATIONS

Plates

Figures

TABLES

SIMULATION OF THE EFFECTS OF PROPOSED CONSTRUCTION OF TWELFTH STREET EXTENSION AND OF FLOOD-PLAIN REFORESTATION ON FLOOD ELEVATIONS, CONGAREE RIVER NEAR COLUMBIA, SOUTH CAROLINA

By R. Erik Schuck-Kolben and Stephen T. Benedict

ABSTRACT

A two-dimensional, depth-averaged, finite-element model was used to study the hydraulic effects of a proposed highway (Twelfth Street Extension) and the planting of pine trees in the flood plain on water-surface elevations associated with the 100- and 500-year flood discharges in a 6.1-mile reach of the Congaree River near Columbia, South Carolina. A six-lane, divided, earthen highway embankment (Interstate 326) with four bridge openings traverses the flood plain in the study area. The rapid expansion of the flood plain of the Congaree River upstream from the highway, a dike system on the left bank, and highly variable roughness combine to cause significant variations in lateral velocities and stage, necessitating the use of a two-dimensional model to accurately evaluate the effects of flood-plain development.

For the 100-year (364,000 cubic feet per second) and the 500-year (630,000 cubic feet per second) discharges, four flood-plain conditions were simulated to determine the effects of the proposed roadway and flood-plain pine tree growth. The flood-plain conditions included: (1) open fields in the flood plain (existing conditions) without Twelfth Street Extension, (2) open fields in the flood plain with Twelfth Street Extension, (3) semi-mature pine trees in the flood plain with Twelfth Street Extension, and (4) mature pine trees in the flood plain with Twelfth Street Extension. The effects of the semi-mature and mature pine trees were determined with the Twelfth Street Extension in place because the Extension causes very little backwater. The effects of the embankment of the proposed Twelfth Street Extension were determined without the pine trees in the flood plain (open fields) because these were the existing conditions prior to reforestation.

Simulations indicate that the trees would retard flood-plain flow, causing water-surface elevations to be higher than normal, and would concentrate flow in the main channel, especially at the Interstate 326 crossing. Model simulations also indicated that maximum backwater produced for the 500-year discharge by semi-mature trees in the flood plain would be 4.1 feet in the flood plain and 3.7 feet in the main channel at river mile 172.25, a location approximately 2 miles upstream of the Interstate 326 crossing. In this simulation, backwater one bridge-width upstream of the Interstate 326 crossing in the main channel was 1.1 feet. For the condition of semi-mature pine trees and the 100-year discharge, backwater one bridge-width upstream of the Interstate 326 crossing in the main channel was 0.8 foot.

Simulation of flood elevations for the condition of open fields in the flood plain and the 500-year discharge indicates that approximately 42 percent of the flow passes through the main-channel bridge and 58 percent passes through the other three bridges. With maximum effect of pine tree growth in the flood plain, flow in the main channel is increased to approximately 55 percent of total flow, and flow through the other three bridges is decreased to 45 percent. Average velocities in the bridge openings for the 500-year flood with open fields in the flood plain ranged from 3.6 to 7.4 feet per second, with a maximum vertically averaged velocity of 9.3 feet per second near the center of the main channel. For the 500-year flood with semi-mature pine trees in the flood plain, average velocities in the bridge openings ranged from 2.9 to 9.5 feet per second, with a maximum vertically averaged velocity of 11.9 feet per second near the center of the main channel. For the 100-year flood discharge and open fields in the flood plain, approximately 50 percent of the flow passes through the main-channel bridge and 50 percent passes through the flood-plain bridges. Semi-mature trees in the flood plain cause the main-channel flow to increase to approximately 63 percent and the flood-plain flow to decrease to 37 percent of the total flow. No significant effect on water-surface elevations or velocities occurs as a result of the construction of the Twelfth Street Extension for either the 100-year or 500-year floods.

INTRODUCTION

The U.S. Geological Survey in cooperation with the South Carolina Department of Highways and Public Transportation conducted an investigation to determine the hydraulic effects of the pine tree growth in the flood plain, and completion of a proposed extension of Twelfth Street on the 100- and 500-year water-surface elevations in the Congaree River flood plain in the vicinity of Interstate 326 (hereafter referred to as I-326) crossing near Columbia.

The I-326 highway, in the study area, traverses a broad flood plain and consists of long stretches of earthen embankments with a number of strategically placed openings for the passage of flood water. These long stretches of earthen embankment impede flow during large floods and cause significant amounts of backwater. Flow of water at high-river stages in wide flood plains generally exhibits a two-dimensional flow pattern, especially in the vicinity of a highway embankment. In addition, flow patterns are affected by changes in channel and flood-plain roughness due to variability of vegetation. Therefore, a two-dimensional, finite-element flow model was used to study the hydraulic effects of the conditions to be evaluated.

In this report, the words "right" and "left" refer to positions that would be reported by an observer facing downstream. Elevations are referenced to sea level.

Purpose and Scope

This report describes the results of a study to evaluate the effects of a proposed highway embankment and reforestation of the flood plain on water-surface elevations, flow velocities, and distribution of discharge through the I-326 bridges in the Congaree River flood plain near Columbia, S.C. for the 100- and 500-year flood discharges. The analysis included the simulation of flow where both longitudinal and lateral variations in velocity and water-surface elevation exist. The effect of highway crossings as well as selected stages of pine tree growth on flood flows was examined by using the Finite Element Surface Water Modeling System (FESWMS-2DH) (Froehlich 1989).

The backwater effects of the embankment of the Twelfth Street Extension were determined without pine trees in the flood plain. The backwater effects of semi-mature and mature pine trees were determined with the Twelfth Street Extension in place. Pine trees 16 to 20 ft high were considered to be semi-mature, and trees over 40 ft high were considered to be mature.

Water-surface elevations and discharges at various locations in the study area were compared to evaluate the effects of the changes in flood-plain conditions. The model network was identical for all simulations except for interchanging roughnesses associated with pine tree growth, changing elevations to account for the embankment of Twelfth Street Extension, and removing some areas of the network for the 100-year flow computations because they were not inundated by the 100-year flood.

Previous Studies

Two previous studies (Lee and Bennett, 1981; Bennett, 1984) have been made on the Congaree River at the I-326 crossing. The original study by Lee and Bennett (1981) used high-water marks and discharges measured during the 1976 flood on the Congaree River near Columbia to calibrate a two-dimensional model for the study area. The channel geometry for the network was determined from field measurements and U.S. Geological Survey topographic maps. Infrared aerial photography was used to define regions of homogeneous roughness. The study employed a two-dimensional finite-element surface-water model developed by Norton and King (Norton and others, 1973; Norton and King, 1973; King and Norton, 1978) to determine the hydraulic effect of the then newly proposed I-326 Highway embankment on the 100-year flood discharge. This was the first study in which such a model was used to determine effects of a proposed multi-opening highway embankment on flood water-surface elevations. Simulations used the 100-year discharge to assess the effects of the proposed highway and various combinations of dikes along the river. The study showed that with the Manning dike in place, the final I-326 design produced a maximum backwater of not more than 0.5 ft for the 100-year discharge of 364,000 ft^3/s.

A second study by Bennett (1984) used a similar network in conjunction with the FESWMS model to describe the hydraulic effects of the proposed I-326 Highway embankment (under construction) and the Manning dike for the 500-year flood discharge of 630,000 ft^3/s. The network boundaries were expanded to accommodate additional inundated area, and the network was then left unchanged for the simulations of the natural and proposed flood-plain conditions. Backwater from the I-326 Highway embankment for the 500-year flood was shown to vary from 0.4 ft at a point upstream of the main channel bridge to 1.2 ft at points along the downstream side of a large island (area of high ground) created during extreme flood stages in the west side of the flood plain as shown in figure 1.

Acknowledgments

The assistance of W.H. Hulbert, South Carolina Department of Highways and Public Transportation, Hydraulics Division; Gerry Schroeder and Norman Snowden, U.S. Department of Transportation, Federal Highway Administration, is gratefully acknowledged.

DESCRIPTION OF THE STUDY AREA

The Congaree River is formed by the confluence of the Broad and Saluda Rivers at Columbia and flows southeastward 51.5 mi (miles) to its confluence with the Wateree River to form the Santee River near the head of Lake Marion shown in the inset of figure 1. The Congaree River is classified as a Piedmont stream because its headwaters, the Broad and Saluda Rivers, originate in the Piedmont in southwestern North Carolina. The area drained by the Congaree River at the Columbia gaging station (station number 02169500), 1.7 mi upstream of the study area, is approximately 7,850 mi^2 (square miles) (fig 1). The greater part of this drainage area (5,240 mi^2) lies in the virtually unregulated Broad River basin. The balance of the drainage area (2,610 mi^2) lies in the highly regulated Saluda River basin.

The study area of the Congaree River discussed in this report is a 6.1-mi reach beginning approximately 3 miles downstream of Columbia (fig. 1) and extending from river mile 167.3 to 173.4. River mile zero is at the mouth of the Santee River. The study reach shown in figure 1 is bounded on the east by the Manning dike, which runs along the east bank of the main channel; on the west by the limits of the flood plain that roughly follow the Seaboard Coast Line Railroad; on the north by the natural constriction of the flood plain; and on the south by an arbitrary line approximately 2 mi downstream of I-326. The flood plain expands rapidly from a width of 0.13 mi at the upstream end of the study area to nearly 4 mi at the I-326 crossing only 3 mi downstream. The streambed generally consists of alluvial sand and finer soils with some large outcrops of rock. About one-half of the flood plain is covered with a combination of dense timber and underbrush. The rest of the flood plain is occupied by cultivated fields interspersed with wooded areas. Some of these fields have been planted with pine trees that will mature about 1998. At the initiation of this study (1988), the pine trees in these fields were at seedling height. At this early stage of development of the trees, it was believed that they would have little effect on flood elevations and might be uprooted by large floods. Therefore, the term "existing conditions" with respect to pine tree growth is used in this report to describe conditions in which the trees, at least from a hydraulic stand-point, are non-existent.

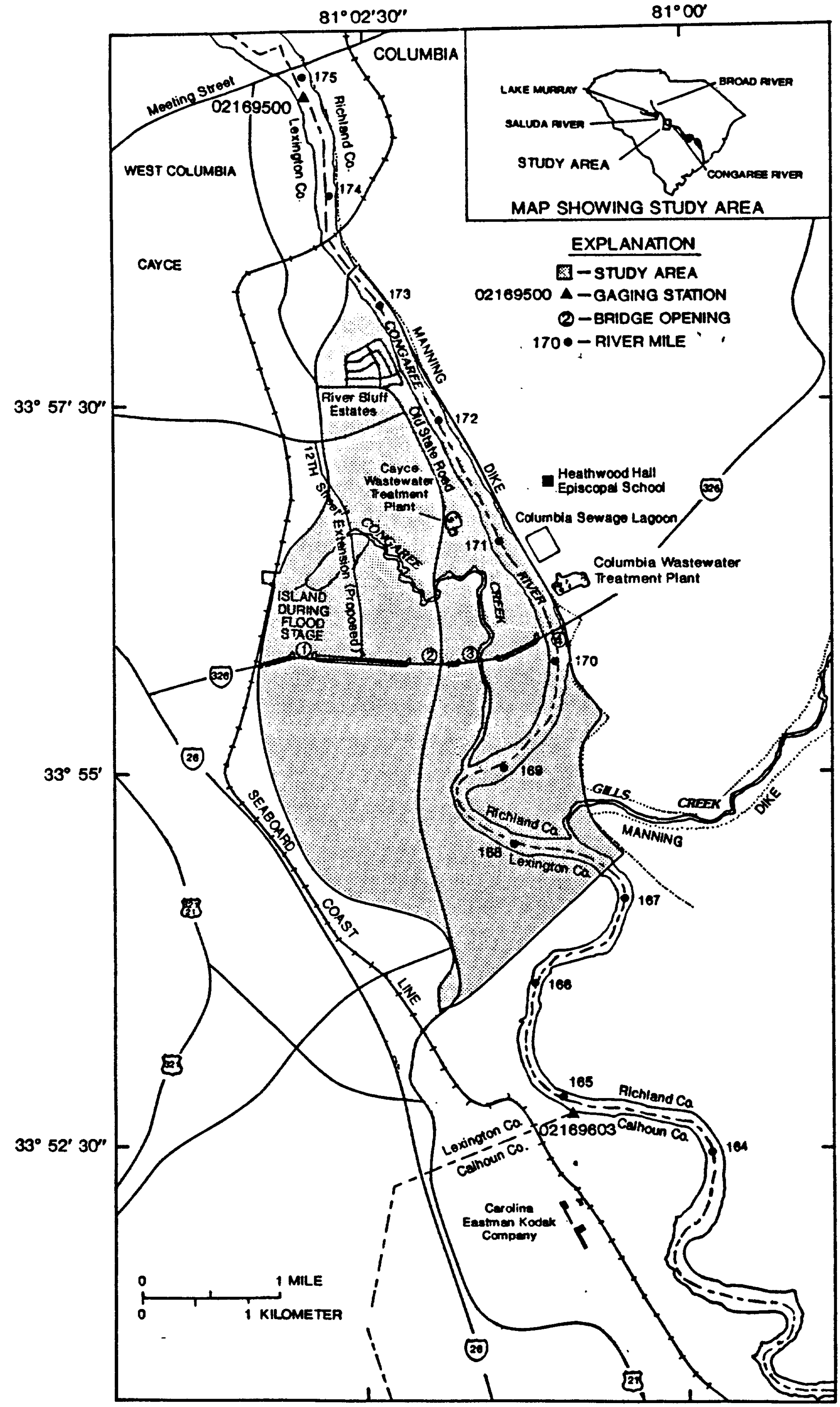

Figure 1—Study area of the Congaree River near Columbia, S.C.

The Manning dike partially protects the limited residential and commercial developments on the left side of the flood plain. Existing developments in the right flood plain include a small housing subdivision at the upper end of the study area and a wastewater treatment plant, 1.1 mi upstream of the I-326 Highway crossing. The wastewater treatment plant is protected by earth dike systems sufficient to divert floods not exceeding the stage of the 100-year flood. The Seaboard Coast Line Railroad tracks were determined to be at an elevation above the water-surface elevations produced by the 500-year flood.

I-326 is part of the Southeastern Beltway system that extends around the south and east sides of Columbia, linking I-26 on the southwest side of Columbia to I-77 and I-20 on the northeast side of the city. I-326 serves as a major crossing of the Congaree River.

The I-326 Highway crosses the Congaree River approximately perpendicular to the direction of flow and is a controlled-access, six-lane, divided highway. The embankment is of earthen construction with four bridge openings as shown in figure 1 and described in table 1. The proposed Twelfth Street Extension will constitute an additional highway embankment within the study area when completed. This proposed extension (fig. 1) will intersect I-326 between the twin-overflow bridges and the twin overpasses over Old State Road and will extend north in the flood plain parallel to the river.

There are seven dual bridges along I-326 in the flood plain. Four of those dual bridges, the main-channel bridge and three flood-relief bridges lie west of the Manning dike. It was assumed for the purpose of this study that the Manning dike would not be breached by flood waters; therefore, the three bridge openings east of the dike were not included in the analysis.

Table 1.--Bridges within the study area for I-326 crossing of the flood plain of the Congaree River

Bridge number	Bridge description	Beginning station	Ending station	Length (feet)
1	Twin overflow bridges	448 + 00	455 + 80	780
2	Twin overpasses over Old State Road (Road S-66)	495 + 10	510 + 10	1,500
3	Twin bridges over Congaree Creek	516 + 25	530 + 65	1,440
4	Twin bridges over the Congaree River	549 + 35	562 + 55	1,320

DISCHARGE AND WATER-SURFACE ELEVATION DATA

Streamflow data for the study area have been collected at the U.S. Geological Survey gaging station (station number 02169500) on the Congaree River at Columbia, S.C., (fig. 1) since October 1939. From October 1891 to December 1933, gage-height records were collected at a site 1,000 ft upstream of the present site. The maximum recorded water-surface elevation at this site was 152.8 ft above sea level, which occurred in August 1908. The peak discharge for that flood as determined from an extension of the rating curve was 364,000 ft^3/s.

The flood of October 11, 1976, had the highest water-surface elevation (142.8 ft above sea level) and associated discharge (155,000 ft^3/s) recorded at the Columbia gage since April 1936. During this flood, a major portion of the flood plain was inundated when a dike along the left side of the main channel was breached. High-water marks for this flood were established by several agencies. The South Carolina Department of Highways and Public Transportation established marks at River Bluff Estates, along Old State Road at Congaree Creek, upstream and downstream from the proposed I-326 crossing, and near the left bank of the main channel at the proposed highway route. High-water marks were also established by the U.S. Army Corps of Engineers at River Bluff Estates, and later by the U.S. Geological Survey at sites near River Bluff Estates, near the Cayce wastewater treatment plant, and at the downstream end of the study reach. A peak water-surface elevation of 127.0 ft above sea level, observed on the staff gage at Congaree River near Cayce, S.C. (station number 02169603) (fig. 1), at the Carolina Eastman Kodak Company[1] (2.5 mi downstream from the study reach), was used to define the lower end of the flood profile.

Two small streams enter the Congaree River in the study reach (fig. 1). Congaree Creek, which empties into the Congaree from the west, has a drainage area of 136 mi^2. Gills Creek, which enters the Congaree from the east just downstream from Congaree Creek, has a smaller drainage area. The maximum recorded discharge of the Congaree Creek at Cayce, S.C. (station number 02169550), of 1,840 ft^3/s occurred on October 1, 1959. These creeks were disregarded in this study because their contributions to flood discharges of the Congaree were relatively small and would occur long before arrival of the Congaree River 100- and 500- year flood peaks.

Flood magnitudes on the Congaree River have been influenced significantly by numerous dams on the Broad and Saluda Rivers. The largest of these structures is Saluda Dam. This Dam is located about 12 miles upstream from the mouth of the Saluda River, and was completed in 1930 by its current operator, the South Carolina Electric and Gas Company[1]. This earth-fill dam forms Lake Murray, which has a surface area of about 51,000 acres at maximum power pool. The lake provides limited flood protection,

---

[1]The use of brand names in this report is for the purpose of identification only and does not constitute endorsement by the U.S. Geological Survey.

hydropower, and recreation for the public. Lake Murray and the Congaree River are the primary sources of water for the city of Columbia and currently supply 10 Mgal/d and 35 Mgal/d of water to the city, respectively.

A discharge-frequency curve was developed for this investigation by the U.S. Army Corps of Engineers on the basis of a log-Pearson type III frequency analysis using recorded flood discharges, and was adjusted for coincidental flood-control storage (fig. 2). A discharge of 630,000 ft^3/s was obtained directly from figure 2 for the 500-year flood. The flood of 1908 (364,000 ft^3/s) was the maximum flood of record and had a peak discharge within 7 percent of the 100-year flood (390,000 ft^3/s) shown in figure 2. Because of the complexity of adjusting the flood frequency for regulations for the period prior to the dams, the SCDHPT, the Federal Highway Administration, and the USGS agreed to use the peak discharge of the flood of 1908 (364,000 ft^3/s) for the peak discharge of the 100-year flood.

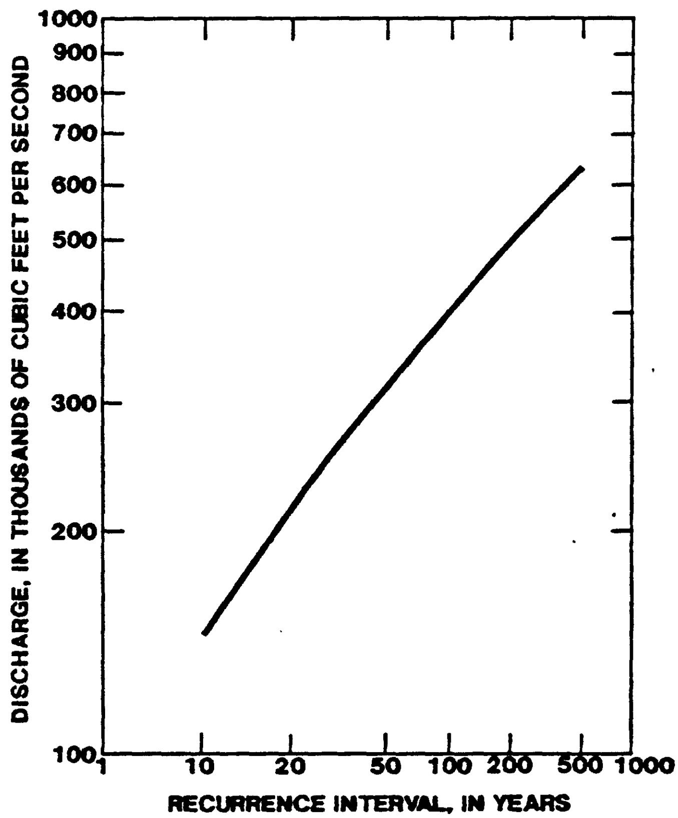

Figure 2--Discharge-frequency relation for the Congaree River at Columbia, S.C. (station number 02169500). (Developed by the U.S. Army Corps of Engineers.)

DESCRIPTION OF MODEL

FESWMS-2DH is a modular set of computer programs developed by Froehlich (1989) specifically for modeling surface-water flows where the flow is essentially two-dimensional in the horizontal plane. The modules of FESWMS-2DH include preprocessing and postprocessing programs in addition to the flow-computation module. Preprocessing programs edit and plot input such as topography, general geometric data, and flow data and arrange them in appropriate formats to be used in the flow model. Postprocessing programs plot maps of velocity vectors and water-surface and backwater contours.

The flow model solves the vertically averaged equations of motion and continuity using a finite-element solution scheme to obtain the depth-averaged velocities and flow depths. A detailed description of the modeling system is beyond the scope of this report; therefore, only the governing equations and a brief outline of the solution scheme are presented.

The version of the model used in this study allows either the Chézy or Manning roughness coefficients to be used for evaluation of friction losses. The Manning coefficient was used in this study. The model elements can be either 6-node triangles or 8- or 9-node quadrilaterals. The effects of turbulence are evaluated by use of an eddy-viscosity formulation. Road overflow and flow through culverts can also be computed by the model.

Flow Equations

The equations that govern the flow of surface water are based on the classical concepts of conservation of mass and momentum. Knowledge of the full three-dimensional flow structure is generally not required for engineering applications, and it is sufficient to use mean-flow quantities in two dimensions. By integration of the three-dimensional equations over the water depth, assuming constant fluid density and a hydrostatic pressure distribution, a set of three equations for two-dimensional flow is obtained. These equations are appropriate for modeling flow in shallow bodies of water. For two-dimensional flow in a horizontal plane, it is convenient to use a right-hand cartesian coordinate system with the x-and y-axis in the horizontal plane and the z-axis directed upward as shown in figure 3. Velocity components in the x, y, and z directions are denoted by u, v, and w, respectively; z_b is the ground-surface or bed elevation, z_a is the water-surface elevation, and H is the depth of flow.

The depth-averaged continuity equation (conservation of mass) is

$$\frac{\partial H}{\partial t} + \frac{\partial}{\partial x} (UH) + \frac{\partial}{\partial y} (VH) = 0 \qquad (1)$$

in which U and V are the depth-averaged values of the horizontal velocities u and v, respectively. The depth-averaged equation of motion (conservation of momentum) in the x-direction is

$$\frac{\partial}{\partial t}(HU) + \frac{\partial}{\partial x}(\alpha_{uu}HUU) + \frac{\partial}{\partial y}(\alpha_{uv}HUV) + gH\frac{\partial}{\partial x}(H + z_b) - \Omega HV - \frac{\rho_a}{\rho} c_w W^2 \cos\psi +$$

$$c_f U (U^2 + V^2)^{1/2} - \frac{\partial}{\partial x}\left[\hat{\nu}H\left(\frac{\partial U}{\partial x} + \frac{\partial U}{\partial x}\right)\right] - \frac{\partial}{\partial y}\left[\hat{\nu}H\left(\frac{\partial U}{\partial y} + \frac{\partial V}{\partial x}\right)\right] = 0 \qquad (2)$$

and the depth-averaged equation of motion in the y-direction is

$$\frac{\partial}{\partial t}(HV) + \frac{\partial}{\partial x}(\alpha_{uv}HVU) + \frac{\partial}{\partial y}(\alpha_{vv}HVV) + gH\frac{\partial}{\partial y}(H + z_b) + \Omega HU - \frac{\rho_a}{\rho} c_w W^2 \sin\psi +$$

$$c_f V (U^2 + V^2)^{1/2} - \frac{\partial}{\partial x}\left[\hat{\nu}H\left(\frac{\partial U}{\partial y} + \frac{\partial V}{\partial x}\right)\right] - \frac{\partial}{\partial y}\left[\hat{\nu}H\left(\frac{\partial V}{\partial y} + \frac{\partial V}{\partial y}\right)\right] = 0 \qquad (3)$$

in which

α_{uu} α_{uv} α_{vv} = momentum correction coefficients (dimensionless),

Ω = Coriolis parameter (radians per second),
g = gravitational acceleration (foot per second squared),
ρ = density of water (slugs per cubic foot),
ρ_a = density of air (slugs per cubic foot),

c_w = wind friction coefficient (dimensionless),

c_f = bottom friction coefficient (dimensionless),

$\hat{\nu}$ = depth-averaged kinematic eddy viscosity (square foot per second),
W = local wind velocity (foot per second),
ψ = angle between the wind direction and the positive x-axis (degrees),
t = time (seconds), and
z_b = bed elevation (feet).

The bottom friction coefficient can be computed either as

$$c_f = g/C^2 \qquad (4)$$

in which C is the Chézy discharge coefficient (foot to the one-half power per second) or as

$$c_f = \frac{gn^2}{2.208\ H^{1/3}} \qquad (5)$$

where n is the Manning roughness coefficient.

The effect of turbulence is evaluated in the model by the Boussenesq eddy-viscosity concept, which assumes the turbulent stresses to be proportional to the mean-velocity gradients. The kinematic eddy viscosity, $\hat{\nu}$, is not a true depth-averaged term in the mathematical sense; however, when related to the bed-shear velocity and local depth, an appropriate depth-averaged stress due to turbulence is obtained. The kinematic eddy viscosity employed by this version of FESWMS-2DH is calculated by using a user-supplied base value of viscosity, and it is increased by the addition of 60 percent of the product of the local depth and bed-shear velocity for each computational point.

For the simulation of steady-state flow in the study reach of the Congaree River, the time derivative terms in equations 1 through 3 were set to zero. The Coriolis force caused by the Earth's rotation was included, but wind friction was deemed negligible and thus was set to zero. Momentum correction coefficients (α_{uu}, α_{uv}, α_{vv}) were all assumed to be one.

Boundary conditions for the set of equations consist of velocity (or unit discharge) components or water-surface elevations at open boundaries and zero-velocity components or zero normal flow at all other boundaries. For a time-dependent simulation, initial conditions must also be specified.

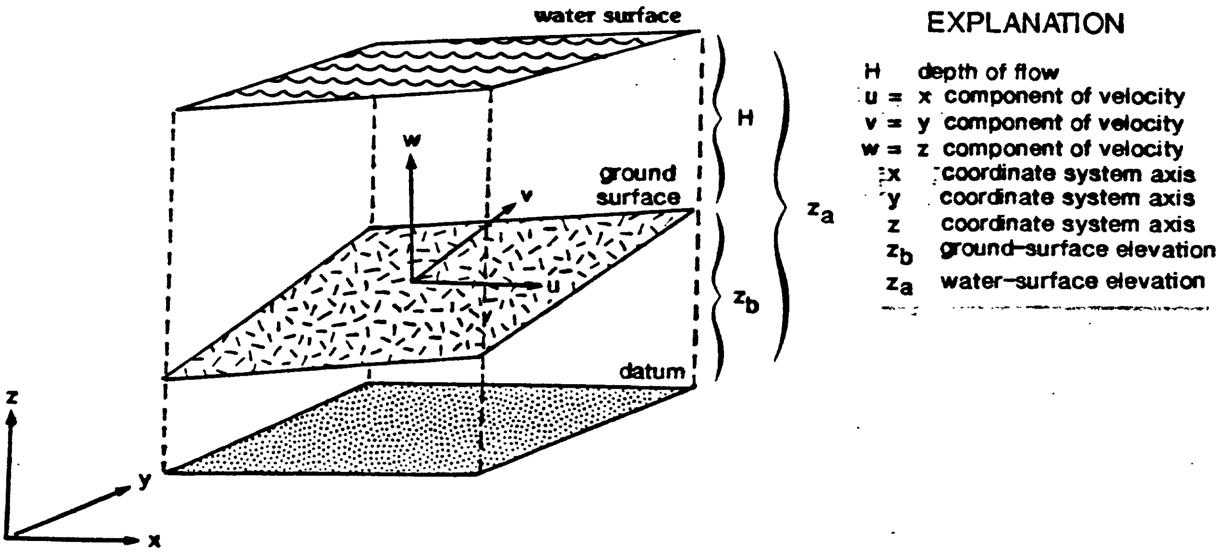

Figure 3--Coordinate system definition.

Weir flow over highway embankments is calculated from the equation for flow over a broad-crested weir

$$Q = C_d LH^{3/2} \tag{6}$$

in which Q is the total discharge over a section of embankment of length L, C_d is a discharge coefficient, and H is the difference in total energy head between the approach (upstream side) of the embankment and the crest elevation of the embankment. If necessary, C_d is adjusted for submergence of the weir.

Numerical Technique

The numerical technique used to solve the governing equations is the Galerkin finite-element method. In this method, the study area is divided into a network of subregions, known as elements, which can either be triangular or quadrangular in shape and can be easily arranged to fit complex boundaries. These elements are defined by a series of node points located at the vertices, side midpoints, and, in the instance of 9-node quadrilaterals, at their centers. Values for the dependent variables can then be uniquely defined within each element in terms of the nodal values by a set of interpolation or shape functions.

Approximations of the dependent variables are then substituted into the governing equations forming a residual, because the equations are usually not satisfied exactly. Weighted averages of the residuals are then computed over the entire solution region using numerical integration. Nodal values of the dependent variables are then computed by requiring that the weighted residual vanishes. In the Galerkin method, the weighting functions are chosen to be the same as those used to interpolate values of the dependent variables within each element.

Newton's iterative method is used to obtain a solution because the system of hydrodynamic flow equations is nonlinear. To apply the Newton method, the governing system of equations, as well as the matrix of derivatives with respect to each of the dependent variables, must be evaluated. This matrix is called the Jacobian or tangent matrix and is computed at each iteration in the solution.

A finite-element network must be carefully designed so that mass is conserved within the system. Improper configuration of elements, especially in channels, can cause mass not to be conserved because only half as many equations are used for conservation of mass as for conservation of momentum in either the x or y direction. The model is capable of computing the discharge across a line (called a continuity-check line) following element sides and beginning and ending at an element vertex. Therefore, conservation of mass can be checked (King and Norton, 1978). Given a well designed network, conservation of mass is obtained for all practical purposes.

A cursory study of continuity-check differences with a two-dimensional finite-element model similar to the one used in this study was completed by Gee and MacArthur (1982). They concluded that the solution was acceptable if the discharge at continuity-check lines does not deviate from the input discharge by more than plus or minus 5 percent.

Finite-Element Network

The finite-element network used for this study was originally developed by Bennett (1984), on the basis of the network of Lee and Bennett (1981), and included the area inundated by the 500-year flood discharge (fig. 4). This area of inundation was approximated by using a one-dimensional step-backwater analysis computed by the U.S. Army Corps of Engineers (COE). This analysis was based on cross-sectional data of the Congaree River and discharges determined from the aforementioned dishcarge-frequency curve. At the time of the COE study, the I-326 Highway embankment had not yet been constructed and was not included in the analysis. Because the COE study reflected natural flood-plain conditions with no significant restrictions of flow, it was concluded that the one-dimensional step-backwater model was sufficient to determine the initial boundaries for the two-dimensional model.

After the boundaries were defined, the study area was divided into a network of triangular elements (Bennett, 1984). Subdivision lines between elements were located where abrupt changes in vegetative cover or topography occurred. Each element was designed to represent an area of nearly homogeneous vegetative cover. Manning roughness coefficients (originally expressed as Chézy discharge coefficients) were assigned to the specific areas of the network and are given in table 2. The finite-element network overlaid on a map of the study area is shown on plate 1, with areas shaded to indicate similar roughness. A more detailed network was used in areas where velocity, depth, and water-surface gradients were expected to be large, such as near bridge openings and in the main-channel areas, in an effort to better describe the hydrodynamics. Some approximations were made to minimize the total number of elements in the system while maintaining the desired accuracy. The network contains approximately 940 elements and 2,300 computational node points, requiring the simultaneous solution of nearly 4,800 nonlinear-algebraic equations.

For this study, the network designed to represent the area inundated by the 500-year flood was used in all simulations so that an accurate comparison of the effects of any proposed modifications could be made. The original network developed by Bennett (1984) (fig. 4) was updated to agree with existing conditions. Flows over the I-326 and Twelfth Street Extension embankment were not allowed by Bennett (1988), because the earlier version of the 2-D model lacked the capability of computing weir flow. However, by use of a newer version of the model in this study, weir flow was computed across I-326 for the 500-year flood. From initial simulations, it was determined that the Twelfth Street Extension embankment did not require weir flow simulation because water-surface elevations were nearly equal on either side of the proposed road, and velocities for both flood discharges in the vicinity of the proposed embankment were also very low. Therefore, flows

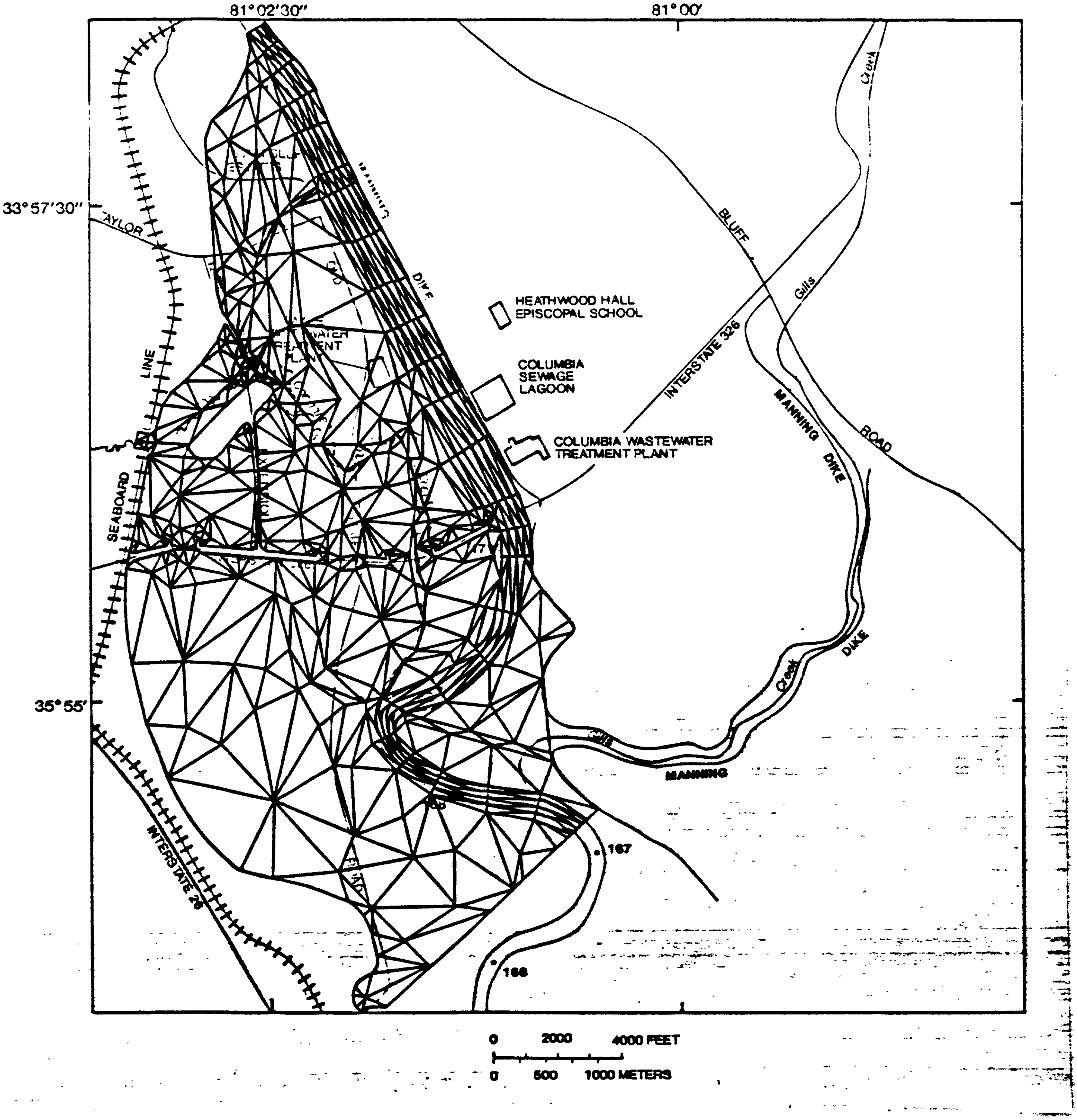

Figure 4--Finite-element network for 500-year simulations (from Bennett, 1984).

Table 2.--Roughness coefficients used in model calibration and simulation

[dashes indicate no data]

Element description or location	Chézy coefficients[a] (foot$^{1/2}$ per second)	Manning's n (second per foot$^{1/3}$)
Main channel of the Congaree River	63.0	0.038
Areas of hardwood forest	20.3	.120
Cleared areas (open grass fields)	89.7	.024
Fully mature pine trees (planted)	--	.125
Semi-mature pine trees (planted)	--	.175
Pavement/shoulder for Twelfth Street Extension	--	.020

[a] Chézy coefficients were determined from calibration of model by Lee and Bennett, 1981.

across the proposed road were modeled by accurately representing the embankment within the finite-element network. Another update consisted of adding a borrow pit that was excavated in 1985 by the local landowner. The borrow pit is on the western and upstream side of the I-326 bridge over the Old State Road (bridge 2). See plate 1.

Boundary Conditions

Discharges for the 100-year and 500-year floods of 364,000 ft^3/s and 630,000 ft^3/s, respectively, were used as input at the upstream boundary. Water-surface elevations, based on a one-dimensional step-backwater analysis for the associated discharge without the I-326 embankment, were used as input for the downstream boundary condition. These downstream water surfaces were assumed to be constant across the width of the downstream boundary and were 133.85 ft and 138.80 ft for the 100- and 500-year floods, respectively. All other boundaries are specified as zero normal-flow boundaries with the exception of embankment sections where weir specifications allow computation of road overflow.

Wier flow was simulated for all 500-year flood runs for the segment of I-326 between bridges 1 and 2 (pl. 1 and fig. 1). The model used the input variables of wier crest elevation, wier discharge coefficient, and wier length in conjunction with computed water depths at the wier to determine the internal boundary conditions for flow at this part of I-326.

The solution for the two-dimensional model is obtained by an iterative procedure. Initially, the model assumes zero velocity and a constant water-surface elevation at each computational node. Owing to the slope of the flood plain, this procedure would result in negative depths at the upstream end of the model if the correct downstream water-surface elevation was used initially. Hence, the initial downstream water-surface elevation was set at a value high enough to avoid this difficulty. The user then reduces the downstream water-surface elevation for each successive iteration until the specified downstream elevation is attained. Additional iterations were then run until the maximum change in depth between two successive iterations at any computational node was less than 0.1 percent. The satisfaction of this condition denoted the final converged solution.

Channel roughness and flood-plain roughness were initially determined during calibration by Lee and Bennett (1981) and were expressed as Chézy roughness parameters. The Chézy coefficients were converted in this study to Manning coefficients by using the equation:

$$n = (1.49/C)R^{1/6} \tag{7}$$

where n is Manning's roughness coefficient, 1.49 is a conversion constant for inch-pound units, C is the Chézy coefficient, and R is the hydraulic radius of the channel. For "very wide" channels the hydraulic radius approaches the depth (Henderson, 1966). Cursory checks of hydraulic radius for the study area indicated that almost all cross sections were hydraulically very wide; therefore, the depth at each node was used in place of the hydraulic radius for determining the value of Manning's n for the given Chézy coefficient. Manning's n was substituted for Chézy C because it is more widely used by the USGS. Roughness coefficients for the areas of pine tree growth were determined on the basis of a study by Arcement and Schneider (1984). Roughness coefficients used throughout the network are given in table 2.

Non-zero values of eddy viscosity are necessary for convergence of the flow model. In previous studies, and in previous versions of the flow model, dynamic eddy viscosity was used, but dynamic eddy viscosity has been replaced in this study by the kinematic eddy viscosity. The equation for converting dynamic eddy viscosity to kinematic eddy viscosity is

$$\hat{\nu} = \eta/\rho \tag{8}$$

where $\hat{\nu}$ is the kinematic eddy viscosity, η is the dynamic eddy viscosity, and ρ is the density of the fluid. Values of the dynamic eddy viscosity used in previous studies were constant at 100 lb-s/ft^2, which correspond almost identically to the base value of kinematic eddy viscosity used in this study of 50 ft^2/s (=97 lb-s/ft^2). The earlier versions of the flow model used a constant value of eddy viscosity throughout the network. The most recent version (used in this study) uses a given base value increased by a given percentage of the product of the bed-shear velocity and depth for each computational point in the network. Numerical experiments in previous studies by Lee and Bennett (1981) and by Gilbert and Schuck-Kolben (1986) showed that solutions using higher values of eddy viscosity produce a system with greater numerical stability. The use of high values of eddy viscosity tends to overestimate energy losses due to turbulence. Therefore, a high initial eddy viscosity is slowly reduced through several iterations to as low a value as possible to attain a numerically stable, converged solution.

Bennett (1984) indicated that part of I-326 would be inundated under the 500-year flood conditions, and that the section of highway between the west overflow bridge and the Old State Road bridge (bridges 1 and 2) would act as a broad-crested weir (fig. 1). Weir crest elevations were set equal to the finished embankment road-surface crown elevation, and weir-discharge coefficients were selected from values that are reasonable for paved roadways (Hulsing, 1967, p. 27).

MODEL CALIBRATION

High-water marks of the October 1976 flood were used to calibrate the flow model (Lee and Bennett, 1981). An assumption was made that although the Manning dike along the left side of the channel was breached in several places during this flood, the outflows were insignificant and could be ignored for steady-state modeling purposes (Lee and Bennett, 1981). Therefore, the dike was treated as a tangential flow boundary. A plot of the finite-element network used for calibration is shown in figure 5. Boundary conditions for calibration were an upstream discharge of 155,000 ft^3/s and a downstream water-surface elevation of 129.2 ft above sea level.

After convergence, values of the roughness coefficients were adjusted until the computed water-surface elevations matched the observed high-water marks as closely as possible. The values of the roughness coefficients (table 2) used in the calibration (Lee and Bennett, 1981) are the same as those used for this study except for the areas of pine tree growth, and the Twelfth Street embankment.

SIMULATION OF EXISTING FLOOD-PLAIN CONDITIONS

As in previous studies (Lee and Bennett, 1981; and Bennett, 1984), the Manning dike was treated as an unbreached boundary. The Manning dike forces the flow to remain on the west side of the flood plain and causes the I-326 embankment to create more backwater than would otherwise exist if flow were allowed onto the east side of the flood plain.

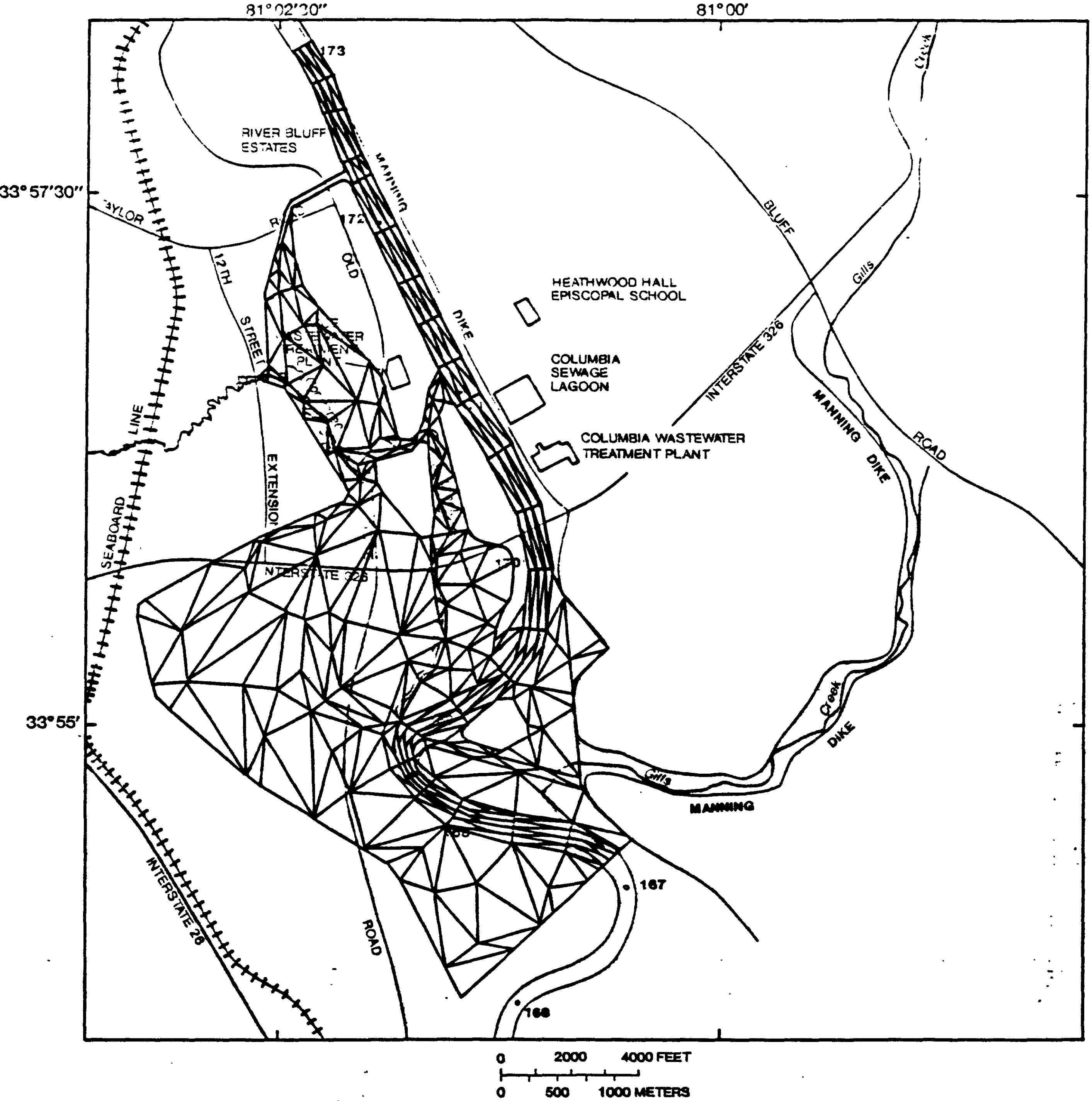

Figure 5--Finite-element network used for calibration, October 1976 flood. (From Lee and Bennett, 1981)

The existing flood-plain and I-326 embankment condition is the same as those simulated in earlier studies, with the exception that a borrow pit was excavated by the local property owner on the upstream right side of the Old State Road bridge opening. The existing-condition model network used for both the 100- and 500-year flood simulations is shown on plate 1. The model network was modified slightly in several places in terms of element-shape configuration to reduce numerical errors. The allowance of weir flow for the segment of the I-326 embankment between bridges 1 and 2 required that the network be modified upstream and downstream of the highway in this area so that element corner nodes aligned across the highway. Also, the elements along the Twelfth Street Extension were modified to more accurately represent the road embankment. Shaded areas on plate 1 indicate fields where pine tree growth was simulated.

The velocity-vector field and water-surface elevation contours for the existing condition (without pine trees in the flood plain and without Twelfth Street Extension) and the 100-year flood discharge are shown on plate 2. For this simulation an upstream discharge of 364,000 ft^3/s and a downstream water-surface elevation of 133.85 ft above sea level were specified as boundary conditions. The velocity-vector field and water-surface elevation contours for the existing condition and the 500-year flood discharge of 630,000 ft^3/s is shown on plate 3. The downstream boundary condition for this simulation is a water-surface elevation of 138.80 ft above sea level. The boundary conditions cause apparent inconsistencies in the velocity vectors and water-surface elevations near the study limits at mile 168 and 173, but have little effect on the areas of primary interest. Discharges were computed for each of the four bridge openings in the I-326 embankment using the model's continuity-check option. This flow distribution for the I-326 crossing for existing conditions is given in table 3. As described earlier, the model does not entirely conserve mass. Flows in table 3 show that mass is conserved within 1.3 percent, which is entirely acceptable for the purposes of this study.

Water-surface elevations and discharges for each of the I-326 bridge openings were extracted from model results. This data, in combination with the SCDHPT road plans, were used to determine the cross-sectional area, average velocity, and average water-surface elevation at each bridge opening (table 3). Maximum vertically averaged velocities at each opening and at the edges of each opening are also given in table 3.

SIMULATION OF THE EFFECTS OF THE PROPOSED TWELFTH STREET EXTENSION

The proposed Twelfth Street Extension was modeled in such a way as to isolate its effect on water-surface elevations for both the 100- and 500-year flood discharges. As discussed later in this report, simulations of flow with mature and semi-mature pine trees in the flood plain shows that the increased roughness due to the trees retards the velocities in the flood plain and causes more flow to remain in the main channel. Therefore, it was concluded that the most pronounced hydraulic effect of the proposed Twelfth Street Extension would exist when flood-plain flow is at a maximum. This maximum-flow condition exists when trees are not present to retard flow in

Table 3.--Hydraulic properties of the bridges across the flood plain of the Congaree River for the 100- and 500-year floods with open fields in the flood plain and without Twelfth Street Extension

[dashes indicate no data]

Bridge number	Bridge description	Discharge (cubic feet per second)	Discharge (percent of total discharge)	Cross-sectional area (square feet)	Average velocity (feet per second)	Maximum vertically averaged velocity (feet per second)	Vertically averaged velocity at left edge (feet per second)	Vertically averaged velocity at right edge (feet per second)	Average water-surface elevation above sea level (feet)
		100-Year flood: Total discharge - 364,000 cubic feet per second							
1	Twin overflow bridges	30,100	8.3	6,580	4.6	5.2	2.2	3.2	139.7
2	Twin overpasses over Old State Road (Road S-66)	85,600	23.5	18,130	4.7	6.5	3.8	3.3	139.8
3	Twin bridges over Congaree Creek	61,300	16.8	24,000	2.6	4.1	1.2	3.8	139.9
4	Twin bridges over the Congaree River	182,000	50.1	28,860	6.3	7.8	1.7	1.3	140.0
		500-Year flood: Total discharge - 630,000 cubic feet per second							
1	Twin overflow bridges	65,200	10.4	10,860	6.0	6.9	1.9	4.7	145.5
	Road overflow, I-326 section between bridges 1 and 2	24,600	3.9	--	--	--	--	--	--
2	Twin overpasses over Old State Road (Road S-66)	156,000	24.7	26,640	5.9	8.3	5.1	3.4	145.6
3	Twin bridges over Congaree Creek	115,000	18.2	32,150	3.6	5.8	1.9	5.8	145.7
4	Twin bridges over the Congaree River	265,000	42.0	35,880	7.4	9.3	2.4	2.2	145.7

the flood plain; therefore, flows for semi-mature and mature pine trees in the flood plain were not simulated in determining the effects of the Twelfth Street Extension on flow patterns.

The ground-elevations and flood profiles, from I-326 to the island (area of high ground), along the proposed Twelfth Street Extension, are shown in figure 6. For both the 100- and 500-year flood events, Twelfth Street Extension causes slight increases in the water-surface elevation (0.05 ft or less) that are strictly confined to the area of the road embankment. Because these small increases round to zero and because the areal extent of the increases are within the embankment limits, it is concluded that this part of the proposed Twelfth Street Extension will cause no backwater for the 100- and 500-year flood events.

However, the part of the Twelfth Street Extension that extends from just north of the island to the north-west boundary of the model does appear to cause a small amount of backwater, 0.1 ft and 0.2 ft for the 100- and 500-year floods, respectively. Lines of equal backwater that would be produced by the north-west part of the proposed embankment for the 100- and 500-year flood discharges are shown in figures 7 and 8. A probable explanation for this increase in water-surface elevation is that the proposed embankment diverts some discharge that was originally flowing west of the island and forces it to flow east of the island. The continuity-checks for the conditions of open fields without Twelfth Street and open fields with Twelfth Street show that there is a diversion of flow from west to east of the island of approximately 1,400 ft^3/s and 5,000 ft^3/s for the 100- and 500-year flood discharges, respectively. This additional flow east of the island will require an increase in the energy needed to drive the discharge through its downstream flow path and this increase in energy is reflected by an increase in the water-surface elevation.

Although the proposed Twelfth Street Extension does produce small changes in the flow patterns and water-surface elevations from those for existing conditions, these changes are small enough to be considered hydraulically insignificant. Thus, for all practical purposes, the conditions of open fields without Twelfth Street and open fields with Twelfth Street Extension are considered hydraulically equivalent for the given flood discharges. Therefore, simulations to compare effects of pine trees in the flood plain were made only with the Twelfth Street Extension in place.

SIMULATION OF THE EFFECTS OF REFORESTATION OF THE FLOOD PLAIN

Pine trees have been planted recently in some areas of the flood plain that were previously open, cultivated fields (plate 1). The trees are loblolly pines, which can reach a height of 70 ft or greater when mature, with trunks as large as 24 inches in diameter. When semi-mature, they are densely foliated, with branches beginning as low as 4 ft from the ground. The trees were planted in a square-grid pattern 6 ft on a side.

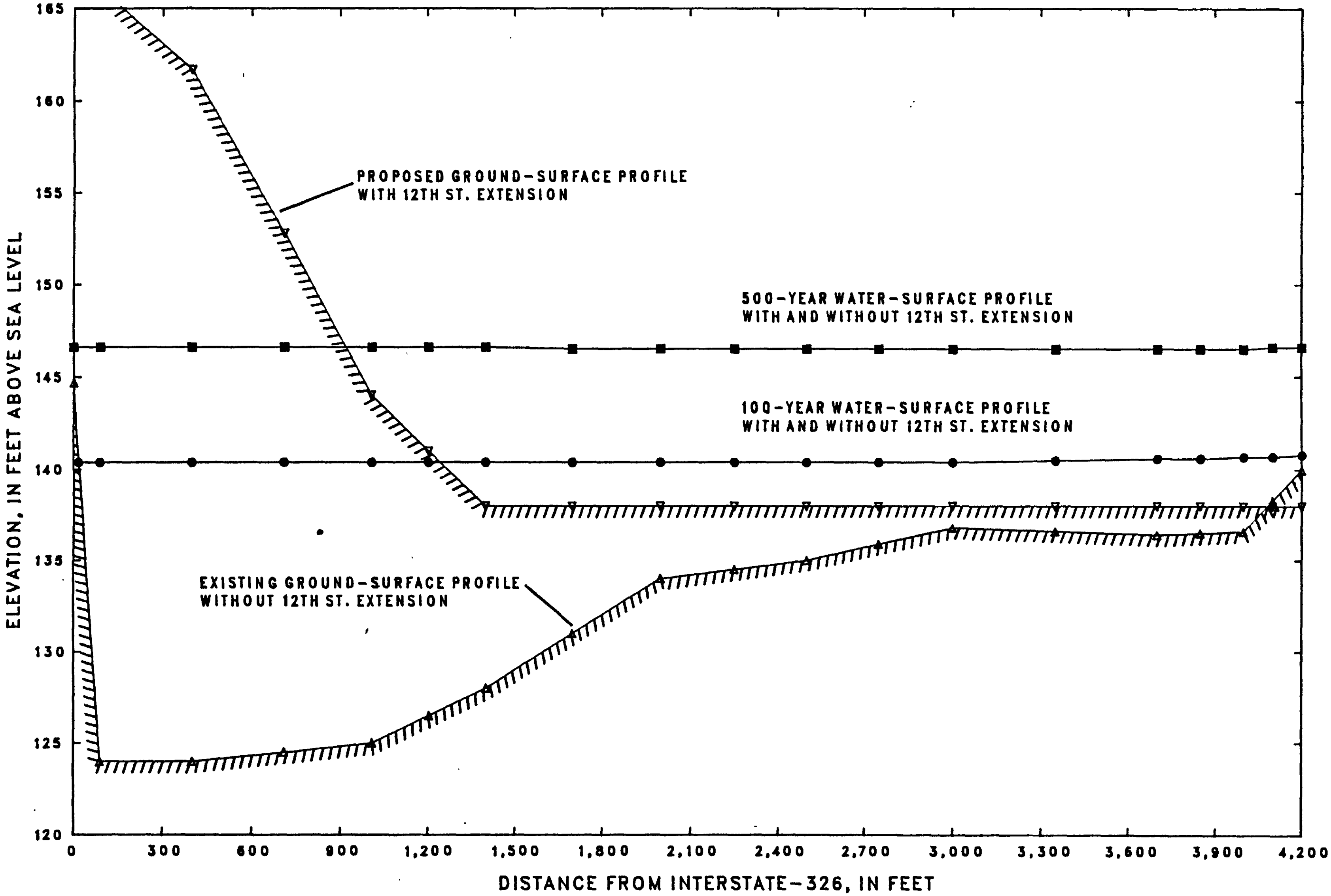

Figure 6--Longitudinal water-surface and ground-surface profiles in the vicinity of the proposed Twelfth Street Extension for the 100- and 500-year discharges.

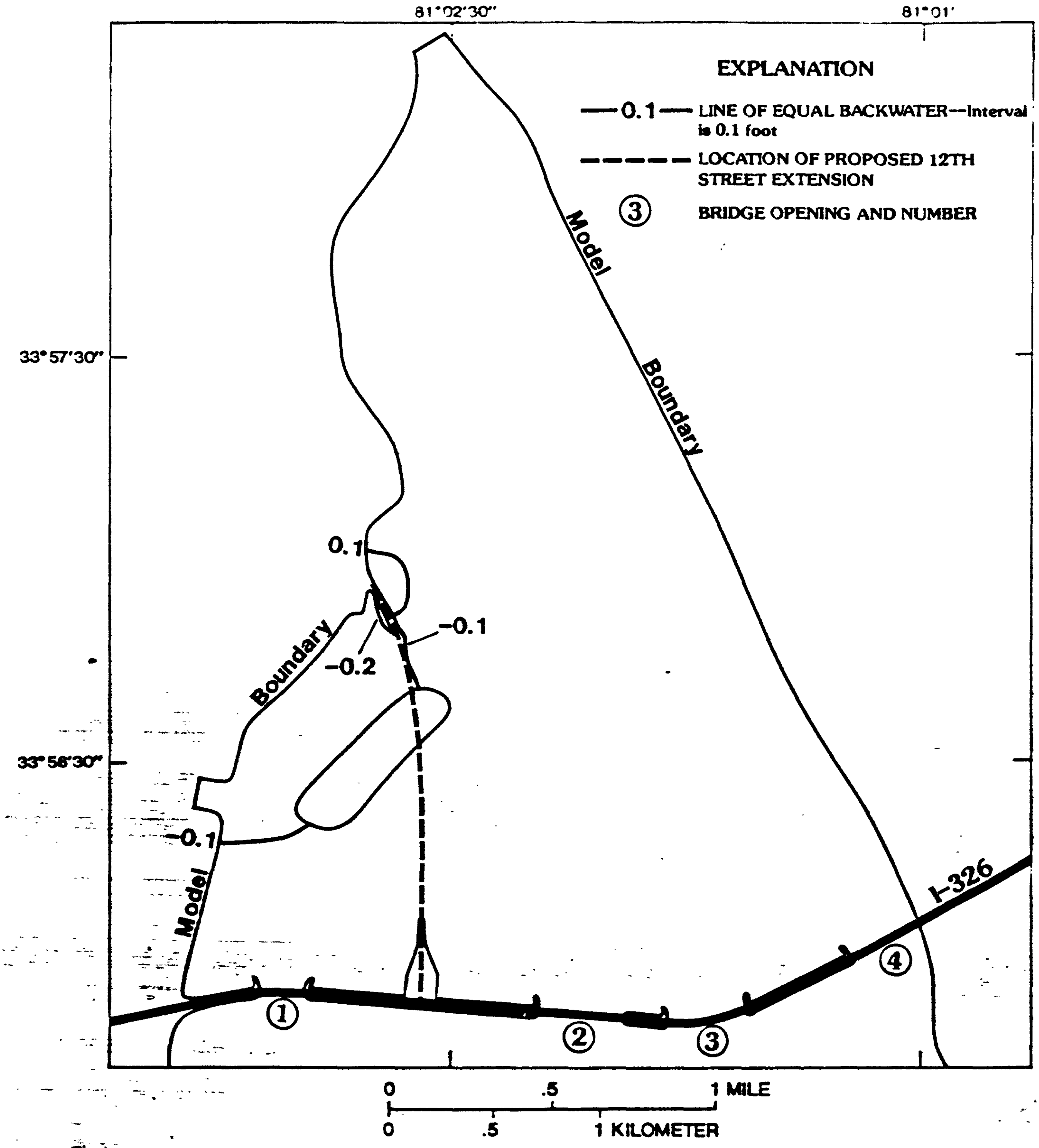

Figure 7--Lines of equal backwater produced by the proposed Twelfth Street Extension for the 100-year flood, Congaree River near Columbia, S.C.

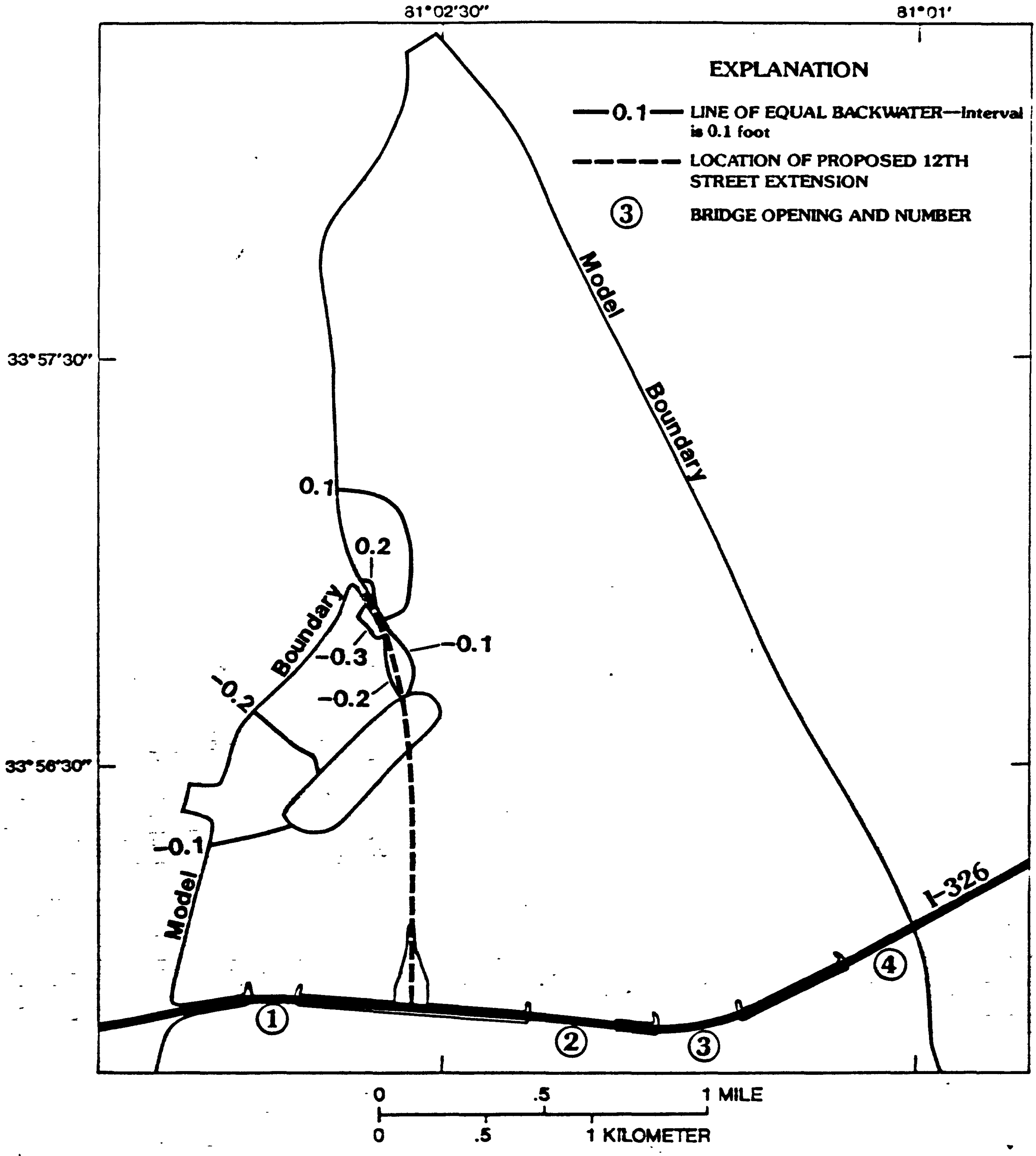

Figure 8--Lines of equal backwater produced by the proposed Twelfth Street Extension for the 500-year flood, Congaree River near Columbia, S.C.

Table 4.--Hydraulic properties of the bridges across the flood plain of the Congaree River for the 100- and 500-year floods with open fields in the flood plain and with Twelfth Street Extension

[dashes indicate no data]

Bridge number	Bridge description	Discharge (cubic feet per second)	Discharge (percent of total discharge)	Cross-sectional area (square feet)	Average velocity (feet per second)	Maximum vertically averaged velocity (feet per second)	Vertically averaged velocity at left edge (feet per second)	Vertically averaged velocity at right edge (feet per second)	Average water-surface elevation above sea level (feet)
			100-Year flood: Total discharge - 364,000 cubic feet per second						
1	Twin overflow bridges	28,900	7.9	6,580	4.4	5.0	2.1	3.0	139.7
2	Twin overpasses over Old State Road (Road S-66)	86,000	23.6	17,980	4.8	6.6	3.8	3.4	139.7
3	Twin bridges over Congaree Creek	61,400	16.9	24,000	2.6	4.1	1.2	3.8	139.9
4	Twin bridges over the Congaree River	183,000	50.3	28,860	6.3	7.9	1.7	1.3	140.0
			500-Year flood: Total discharge - 630,000 cubic feet per second						
1	Twin overflow bridges	64,600	10.3	10,860	6.0	6.9	1.8	4.7	145.5
	Road overflow, I-326 section between bridges 1 and 2	23,800	3.8	--	--	--	--	--	--
2	Twin overpasses over Old State Road (Road S-66)	157,000	24.9	26,640	5.9	8.3	5.1	3.6	145.6
3	Twin bridges over Congaree Creek	114,000	18.2	32,150	3.6	5.8	1.9	5.8	145.7
4	Twin bridges over the Congaree River	266,000	42.2	35,880	7.4	9.3	2.4	2.3	145.7

A hydraulic analysis used computed hydraulic data from the existing-condition simulations, in terms of velocity and depth in areas where trees were planted, in order to determine hydraulic parameters necessary for accurate computation of roughness coefficients. The semi-mature trees were determined to have the most effect on the flow because of their dense and low foliage. A roughness coefficient (Manning's n) of 0.175 for areas of semi-mature pine trees was estimated by an analytical technique developed by Arcement and Schneider (1984). The mature pine trees have less effect on the flow because their lowest branches are above the elevation of the water surface. Simulations of both floods also were carried out by using a roughness coefficient of 0.125 for areas of mature pine trees.

Although it has been shown that the effect of the proposed Twelfth Street Extension will be negligible, the proposed highway was included in all simulations made to determine the effect of the trees. The hydraulic effect of mature and semi-mature pine trees is similar for both the 100- and 500-year floods. The water-surface elevations are increased and a greater percentage of total discharge is confined to the main channel. Water-surface contours and velocity vectors for the conditions of open fields, semi-mature, and mature pine trees are shown in plates 4, 6, and 10, respectively, for the 100-year discharge and plates 5, 7, and, 11 respectively, for the 500-year discharge. The hydraulic effect produced by an increase in discharge with similar roughnesses is demonstrated by comparison of plates 4 and 5 with no trees, and plates 6 and 7 for conditions of maximum roughness produced by semi-mature trees. Plates 6 and 7 show water-surface contours and velocity vectors for the 100-and 500-year discharges, respectively, with roughness coefficients of 0.175 for areas of semi-mature pine trees. Plate 10 and 11 show the water-surface contours and velocity vectors for the 100- and 500-year discharges, with the condition of mature pine trees. Hydraulic conditions for the bridge openings with the maximum effect of pine trees are given in table 5. Hydraulic data for the bridge openings with the condition of mature pine trees (Manning's roughness coefficient of 0.125) are given in table 6.

The most obvious effect of reforestation is seen by comparison of plates 4 and 6 for the 100-year flood and plates 5 and 7 for the 500-year flood. This comparison indicates that the growth of trees near the northern boundary of the study area causes considerably more flow to remain in the main channel. The continuity-check option was used to determine discharges across the flood plain and main channel near the center and immediately below the northern tree-growth area shown in plate 1. In the middle of the upstream tree-growth area, main-channel discharge for the 100-year flood (total discharge 364,000 ft^3/s) increased from 251,000 ft^3/s with open fields to 272,000 ft^3/s with semi-mature trees. Downstream of the planted area, main-channel discharge for the 100-year flood increased from 203,000 ft^3/s with open fields to 256,000 ft^3/s with semi-mature trees. Downstream of the planted area, main-channel discharge for the 500-year discharge (total discharge 630,000 ft^3/s) increased from 289,000 ft^3/s with open fields to 383,000 ft^3/s with semi-mature pine trees.

Table 5.--Hydraulic properties of the bridges across the flood plain of the Congaree River for the 100- and 500-year floods with semi-mature pine trees in the flood plain and with Twelfth Street Extension

[dashes indicate no data]

Bridge number	Bridge description	Discharge (cubic feet per second)	Discharge (percent of total discharge)	Cross-sectional area (square feet)	Average velocity (feet per second)	Maximum vertically averaged velocity (feet per second)	Vertically averaged velocity at left edge (feet per second)	Vertically averaged velocity at right edge (feet per second)	Average water-surface elevation above sea level (feet)
	100-Year flood: Total discharge - 364,000 cubic feet per second								
1	Twin overflow bridges	24,700	6.8	6,800	3.6	4.6	2.6	2.2	140.0
2	Twin overpasses over Old State Road (Road S-66)	59,600	16.4	18,710	3.2	5.9	.8	2.9	140.2
3	Twin bridges over Congaree Creek	51,500	14.2	24,420	2.1	2.6	1.3	1.0	140.2
4	Twin bridges over the Congaree River	231,000	63.4	29,230	7.9	9.9	2.2	1.7	140.3
	500-Year flood: Total discharge - 630,000 cubic feet per second								
1	Twin overflow bridges	50,900	8.1	11,160	4.6	5.9	3.3	3.0	145.9
	Road overflow, I-326 section between bridges 1 and 2	30,800	4.9	--	--	--	--	--	--
2	Twin overpasses over Old State Road (Road S-66)	110,000	17.4	27,380	4.0	6.9	1.4	3.0	146.1
3	Twin bridges over Congaree Creek	95,600	15.2	32,710	2.9	3.6	1.8	1.3	146.1
4	Twin bridges over the Congaree River	345,000	54.7	36,380	9.5	11.9	3.2	3.0	146.1

Table 6.--Hydraulic properties of the bridges across the flood plain of the Congaree River for the 100- and 500-year floods with mature pine trees in the flood plain and with Twelfth Street Extension

[dashes indicate no data]

Bridge number	Bridge description	Discharge (cubic feet per second)	Discharge (percent of total discharge)	Cross-sectional area (square feet)	Average velocity (feet per second)	Maximum vertically averaged velocity (feet per second)	Vertically averaged velocity at left edge (feet per second)	Vertically averaged velocity at right edge (feet per second)	Average water-surface elevation above sea level (feet)
			100-Year flood: Total discharge - 364,000 cubic feet per second						
1	Twin overflow bridges	25,300	6.9	6,730	3.8	4.6	2.7	2.5	139.9
2	Twin overpasses over Old State Road (Road S-66)	66,000	18.1	18,420	3.6	6.2	1.3	3.2	140.0
3	Twin bridges over Congaree Creek	51,600	14.2	24,280	2.1	2.8	1.2	1.2	140.1
4	Twin bridges over the Congaree River	222,000	60.9	29,110	7.6	9.6	2.1	1.6	140.2
			500-Year flood: Total discharge - 630,000 cubic feet per second						
1	Twin overflow bridges	54,100	8.6	11,090	4.9	6.1	3.2	3.6	145.8
	Road overflow, I-326 section between bridges 1 and 2	29,200	4.6	--	--	--	--	--	--
2	Twin overpasses over Old State Road (Road S-66)	122,000	19.4	27,080	4.5	7.3	2.1	3.3	145.9
3	Twin bridges over Congaree Creek	96,500	15.3	32,570	3.0	3.8	1.7	1.9	146.0
4	Twin bridges over the Congaree River	326,000	51.7	36,260	9.0	11.3	3.0	2.8	146.0

The flood plain immediately downstream from the area of tree growth near the northern boundary was divided into two regions of interest with respect to discharge. These two regions are the areas on either side of a large island (area of high ground) shown in plate 1. For the 100-year flood, discharge is reduced from 17,200 ft^3/s with open fields to 16,200 ft^3/s with semi-mature trees. For the 500-year flood discharge, the flow west of the island is increased from 35,800 ft^3/s with open fields to 40,900 ft^3/s with semi-mature pine trees. Flood-plain flow east of the island was reduced for the 100-year discharge from 126,000 ft^3/s with open fields to 88,000 ft^3/s with semi-mature trees. Similarly, for the 500-year discharge, flow east of the island was reduced from 281,000 ft^3/s with open fields to 200,000 ft^3/s with semi-mature pine trees.

Cumulative discharge plotted from west to east along I-326 for the 100- and 500-year discharges, respectively, is shown in figure 9. A reduction of flow at bridges 1 through 3 and an increase in the flow through the main-channel bridge is indicated in figure 9 and tables 4, 5, and 6. Main-channel bridge discharge for the 100-year flood increased from 183,400 ft^3/s with existing flood-plain conditions to 231,000 ft^3/s with semi-mature trees, an increase of about 26 percent. For the 500-year flood, main-channel discharge at the I-326 bridge increased from 266,000 ft^3/s with open fields to 345,000 with semi-mature pine trees (about 30 percent). Maximum vertically averaged velocities for the 100-year flood in the main-channel bridge increased from 7.9 ft/s with existing conditions to 9.9 ft/s with semi-mature pine trees. Maximum vertically-averaged velocities for the 500-year discharge increased from 9.3 ft/s with open fields to 11.9 ft/s with semi-mature pine trees. Vertically-averaged velocities along the left edge of the main channel bridge (Manning dike) for the 100-year flood discharge increased from about 1.7 ft/s with open fields to about 2.2 ft/s with semi-mature pine trees in the flood plain. For the 500-year flood discharge, vertically averaged velocities at the left edge of the main-channel bridge increased from about 2.4 ft/s with open fields to about 3.2 ft/s with semi-mature pine trees.

Backwater is an increase in water-surface elevation above the normal water-surface elevation that can be caused by an encroachment, a downstream reservoir, or a confluence with another stream. Although the encroachment generally is thought to be a structure such as a highway embankment, it can also be vegetation. In this discussion, backwater refers only to the increase in water-surface elevation above that calculated with the I-326 Highway embankment, Manning dike, and Twelfth Street Extension in place. Backwater produced by I-326 was not specifically identified in this study, as the maximum effect of the structure has been studied previously (Lee and Bennett, 1981; and Bennett, 1984).

The maximum effect of the growth of pine trees on water-surface elevations is best shown on plates 8 and 9 by lines of equal backwater produced by the semi-mature trees for the 100- and 500-year flood discharges. These plots were generated by calculating the difference in water-surface elevations between the simulations with semi-mature pine trees and the simulations with open fields (existing conditions). Water-surface elevations for these flood-plain conditions and associated backwater

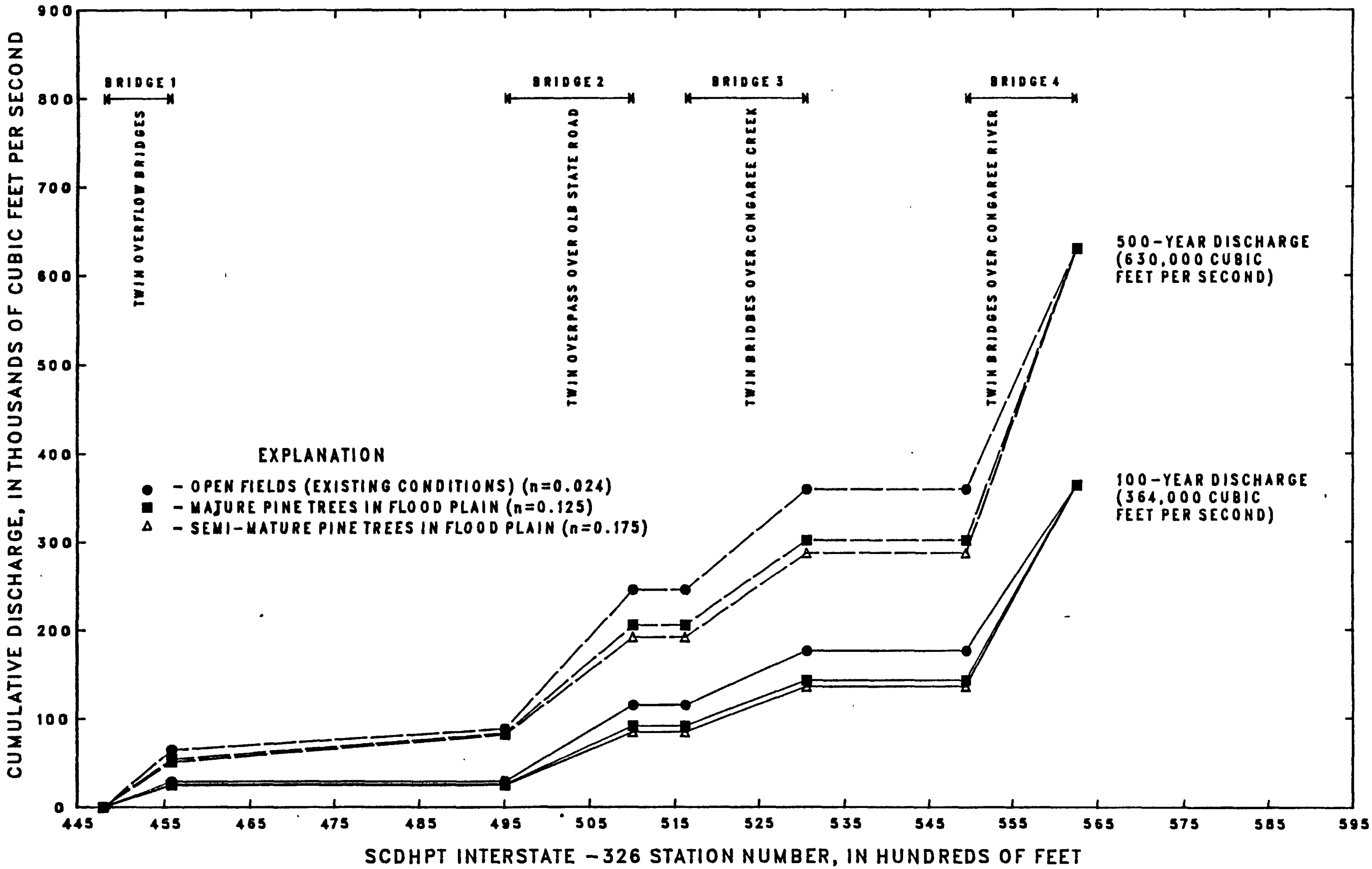

Figure 9--Cumulative discharge from west to east along Interstate-326 embankment for the 100- and 500-year discharges for existing flood-plain conditions and two stages of pine tree growth.

produced for 100- and 500-year discharges at various locations in the flood plain are given in table 7. Main-channel flood profiles based on data at half-mile intervals for existing conditions, and with semi-mature, and mature trees in the flood plain are shown in figure 10. Maximum backwater in the flood plain of 4.1 ft was produced by the semi-mature trees with the 500-year flood at a location 1,000 ft southeast of the south side of the River Bluff Estates subdivision. Maximum backwater in the main channel for this flood condition was 3.7 ft. Maximum backwater at a point in the main channel one bridge width upstream of the twin bridges of I-326 was 1.1 ft.

Table 7.--Water-surface elevations (with and without pine tree growth) and associated backwater produced for 100- and 500-year discharges at various flood-plain and main channel locations with Twelfth Street Extension

Map location reference number	Location	Water surface elevations above sea level (feet)			Backwater (feet)	
		Existing condition n = 0.024	Mature pine trees n = 0.125	Semi-mature pine trees n = 0.175	Mature pine trees n = 0.125	Semi-mature pine trees n = 0.175
		100-Year flood discharge				
1	Location of maximum backwater in main channel	143.6	145.7	146.1	2.1	2.5
2	Location of maximum backwater in flood plain	143.6	146.2	146.6	2.6	3.0
3	Main channel at twin bridges over the Congaree River	140.7	141.3	141.5	.6	.8
4	Cayce wastewater treatment plant	141.8	142.9	143.2	1.1	1.4
5	Seaboard Coast Line bridge over Congaree Creek	141.3	142.1	142.4	0.8	1.1
6	River Bluff Estates, south side	145.1	147.0	147.2	1.9	2.1
7	River Bluff Estates, east side	145.3	147.0	147.3	1.7	2.0

Table 7.--Water-surface elevations (with and without pine tree growth) and associated backwater produced for 100- and 500-year discharges at various flood-plain and main channel locations with Twelfth Street Extension--Continued

Map location reference number	Location	Water surface elevations above sea level (feet)			Backwater (feet)	
		Existing condition n = 0.024	Mature pine trees n = 0.125	Semi-mature pine trees n = 0.175	Mature pine trees n = 0.125	Semi-mature pine trees n = 0.175
	500-Year flood discharge					
1	Location of maximum backwater in main channel	149.3	152.3	153.0	3.0	3.7
2	Location of maximum backwater in flood plain	149.6	152.9	153.7	3.3	4.1
3	Main channel at twin bridges over the Congaree River	146.7	147.5	147.8	.8	1.1
4	Cayce wastewater treatment plant	147.9	149.4	149.8	1.5	1.9
5	Seaboard Coast Line bridge over Congaree Creek	147.2	148.2	148.6	1.0	1.4
6	River Bluff Estates, south side	150.9	153.3	154.1	2.4	3.2
7	River Bluff Estates, east side	151.2	153.5	154.2	2.3	3.0

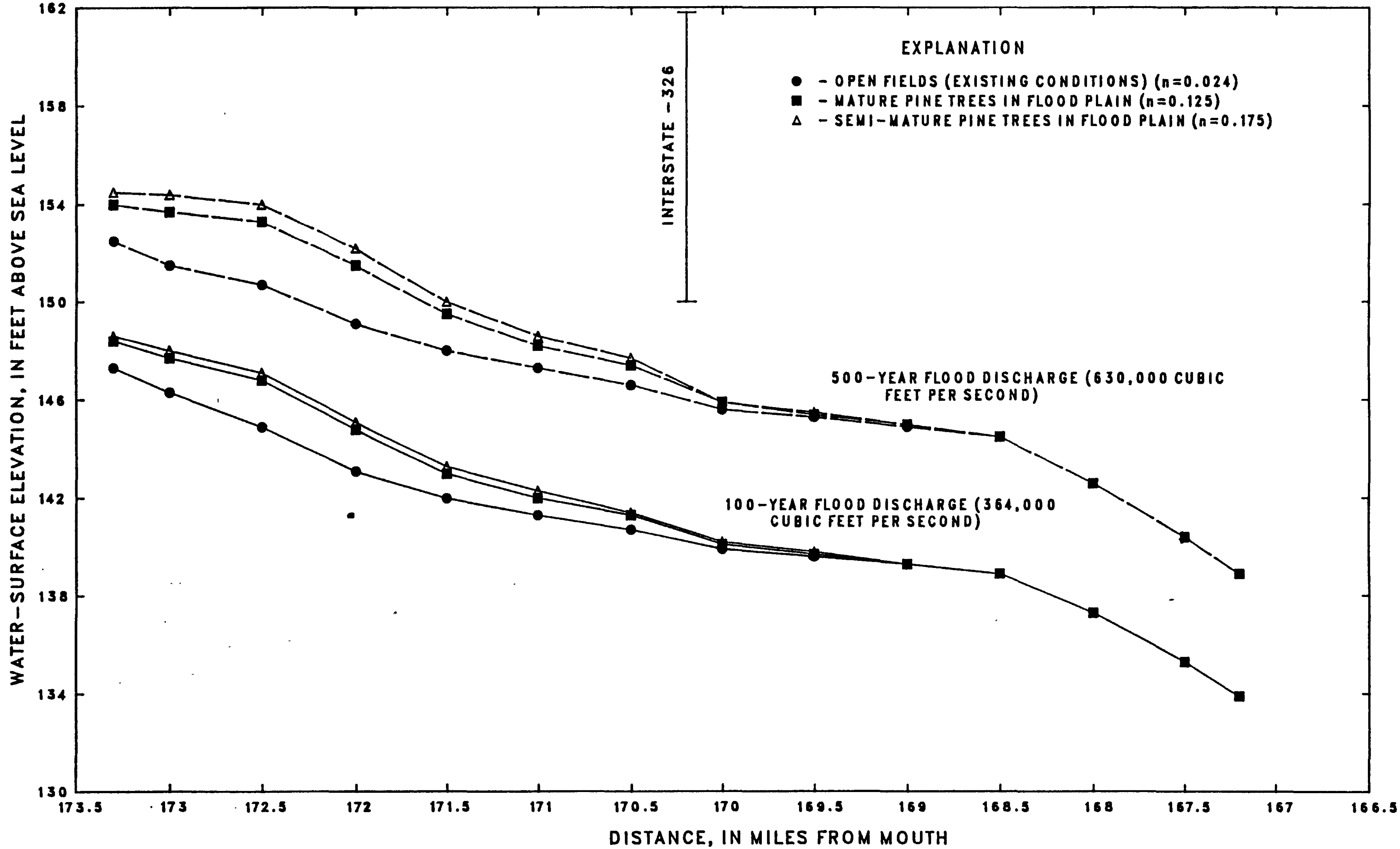

Figure 10--Water-surface profiles for the main channel for the 100- and 500-year discharges for existing flood-plain conditions and two stages of pine tree growth.

SUMMARY

A two-dimensional finite-element model was used to simulate the effects of the proposed Twelfth Street Extension and the reforestation of the Congaree River flood plain at the I-326 crossing near Columbia, S.C., on the flow patterns for the 100- and 500-year discharges. A calibrated finite-element network, which was developed in a previous investigation was updated to reflect existing conditions and to represent the proposed roadway and selected stages of pine tree growth (mature and semi-mature) for the reforestation of the flood plain.

By using the Finite Element Surface Water Modeling System (FESWMS-2DH), flows for four flood-plain conditions were simulated for the 100-year (364,000 ft^3/s) and the 500-year (630,000 ft^3/s) discharges. The effects on water-surface elevations, flow velocities, and flow distributions for the various flood-plain conditions were determined. The flood-plain conditions included: (1) open fields in the flood plain (existing conditions) without Twelfth Street Extension, (2) open fields in the flood plain with Twelfth Street Extension, (3) semi-mature pine trees in the flood plain with Twelfth Street Extension, and (4) mature pine trees in the flood plain with Twelfth Street Extension. The effects of the semi-mature and mature pine trees were determined with the Twelfth Street Extension in place, because the Extension causes very little backwater. The effects of the embankment of the proposed Twelfth Street Extension were determined without the pine trees in the flood plain (open fields), because these were the existing conditions prior to reforestation.

From the simulations, it was concluded that the proposed roadway will have little effect on the flow patterns for the 100- and 500-year discharges. Although changes in the flows are produced by the embankment, they are small enough to be considered hydraulically insignificant. Therefore, it was concluded that for all practicle purposes, the conditions of open fields without Twelfth Street Extension and open fields with Twelfth Street Extension are hydraulically equivalent for the given flood discharges.

In general, the effect of the growth of pine trees was to increase water-surface elevations upstream from the I-326 embankment and to increase flow in the main channel for the 100- and 500-year flood discharges. A combination of semi-mature trees and a 500-year flood condition produced the maximum effect, which was maximum backwater of 4.1 feet in the flood plain at a location 1,000 feet southeast of the south side of the River Bluff Estates subdivision. Maximum backwater in the main channel for this flood condition was 3.7 feet. Maximum backwater at a point in the main channel one bridge width upstream of the the twin bridges of I-326 was 1.1 feet. Discharges through the three overflow bridges were reduced, and flow in the main-channel bridge was increased by as much as 30 percent.

SELECTED REFERENCES

Arcement, G.J., Jr., and Schneider, V.R., 1984, Guide for selecting Manning's roughness coefficients for natural channels and flood plains: U.S. Department of Transportation, Federal Highway Administration, Report No. FHWA-TS-84-204, 68 p.

Bennett, C.S., III, 1984, Impact of the proposed I-326 crossing on the 500-year flood stages of the Congaree River near Columbia, South Carolina: U.S. Geological Survey Water-Resources Investigations Report 84-4280, 23 p.

Bradley, J.N., 1970, Hydraulics of bridge waterways, hydraulic design series No. 1: Federal Highway Administration, Washington, D.C., 111 p.

Chow, V.T., 1959, Open-channel hydraulics: New York, McGray-Hill Book Company, 680 p.

Froehlich, D.C., 1989, Finite-element surface water modeling system: Two dimensional flow in a horizontal plane, user's manual, U.S. Department of Transportation, Federal Highway Administration, Publication number FHWA-RD-88-177, 285 p.

Gee, D.M., and MacArthur, R.C., 1982, Evaluation and application of the generalized finite element hydrodynamic model, RMA-2, in MacArthur, R.C. and others, eds., Two-dimensional flow modeling: U.S. Army Corps of Engineers--Sponsored seminar on two-dimensional flow modeling, Davis, Calif., 1981, Proceedings, Davis, Calif., U.S. Army Corps of Engineers, The Hydrologic Engineering Center, p. 97-109.

Gilbert, J.J., and Froehlich, D.C., 1985, Simulation of the effect of U.S. Highway 90 on Pearl River floods of April 1980 and April 1983 near Slidell, Louisiana: U.S. Geological Survey Water-Resources Investigations Report 85-4286, 47 p.

Gilbert, J.J., and Schuck-Kolben, R.E., 1986, Effects of proposed highway embankment modifications on water-surface elevations in the Lower Pearl River flood plain near Slidell, Louisiana: U.S. Geological Survey Water-Resources Investigations Report 86-4129, 36 p.

Henderson, F.M., 1966, Open-channel flow: New York, MacMillan Publishing Co., Inc., 522 p.

Hulsing, Harry, 1967, Measurement of peak discharge at dams by indirect methods: U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chap. A5, p. 29.

King, I.P., and Norton, W.R., 1978, Recent applications of RMA's finite-element models for two-dimensional hydrodynamics and water quality, in Brebbia, C.A. and others, eds., Finite elements in weather resources: International Conference on Finite Elements in Water Resources, 2d, London, England, 1978, Proceedings, London, Pentech Press, p. 2.81-2.99.

REFERENCES--Continued

Lee, J.K., and Bennett, C.S., III, 1981, A finite-element model study of the impact of the proposed I-326 crossing on flood stages of the Congaree River near Columbia, South Carolina: U.S. Geological Survey Open-File Report 81-1194, 56 p.

Norton, W.R., and King, I.P., 1973, A finite-element model for Lower Granite Reservoir. Computer application supplement and user's guide: Walnut Creek, Calif., Water Resources Engineers, Inc., 90 p.

Norton, W.R., King, I.P., and Orlob, G.T., 1973, A finite-element model for Lower Granite Reservoir: Walnut Creek, Calif., Water Resources Engineers, Inc., 138 p.

Wiche, G.J., Gilbert, J.J., Froehlich, D.C., and Lee, J.K., 1984, Analysis of alternative modifications for reducing backwater at the I-10 crossing of the Pearl River near Slidell, Louisiana: U.S. Geological Survey Water-Supply Paper 2267, 48 p.