SIMULATION OF THE EFFECT OF U.S. HIGHWAY 90 ON PEARL RIVER FLOODS OF APRIL 1980 AND APRIL 1983 NEAR SLIDELL, LOUISIANA

By J. Josh Gilbert, and David C. Froehlich

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ABSTRACT

The effect of U.S. Highway 90 on water-surface elevations in the Pearl River flood plain during the floods of April 1980 and April 1983, was determined on the basis of results from a two-dimensional finite-element surface-water flow model. The model was used to simulate flow in the river flood-plain system for conditions both with and without the U.S. Highway 90 embankments in place. The effect of the highway crossing on flood elevations was determined by the difference in water-surface elevations between the two simulations.

Results of the 1980 flood simulations show that the maximum backwater along the U.S. Highway 90 embankment was 1.0 foot, located on the upstream side of the embankment near the intersection with U.S. Highway 190. The maximum backwater on the west side of the flood plain was also 1.0 foot; while on the east side near Pearlington, Mississippi, it was 0.3 foot. At the Interstate Highway 10 crossing, approximately 4 miles upstream, backwater was 0.2 foot on the east and west edges of the flood plain.

Simulations of the 1983 flood indicate a maximum backwater of 1.2 feet on the upstream side of the highway embankment, midway between the West Pearl River and West Middle Pearl River bridge openings. Maximum backwater on the west side of the flood plain was 1.2 feet near the intersection of U.S. Highway 190 and U.S. Highway 90. Maximum backwater on the east side of the flood plain near Pearlington was 0.2 foot. Upstream of the Interstate Highway 10 crossing, backwater was 0.1 foot at the east and west edges of the flood plain.

INTRODUCTION

During April 1980 and April 1983, extreme flooding of the lower Pearl River caused extensive property damage in subdivisions located on the flood plain in the Slidell, La., area (fig. 1). Many persons were forced from their homes until the flood water receded.

Property damage in the Slidell area caused by the 1980 flood, slightly greater than a 50-year flood, was estimated to be $12 million (U.S. Army Corps of Engineers, 1981, p. 76). The 1980 flood forced the closing of the Interstate Highway 10 (I-10) crossing of the Pearl River flood plain between Slidell, La., and Bay St. Louis, Miss., for several hours.

Damage in the Slidell area caused by the 1983 flood, a 200-year flood (and the largest of record), was estimated to be $16 million (Tom Creaghan, Louisiana State Office of Emergency Preparedness, oral commun., 1983). Again, I-10 was closed, along with U.S. Highway 90 (U.S. 90) and U.S. Highway 190

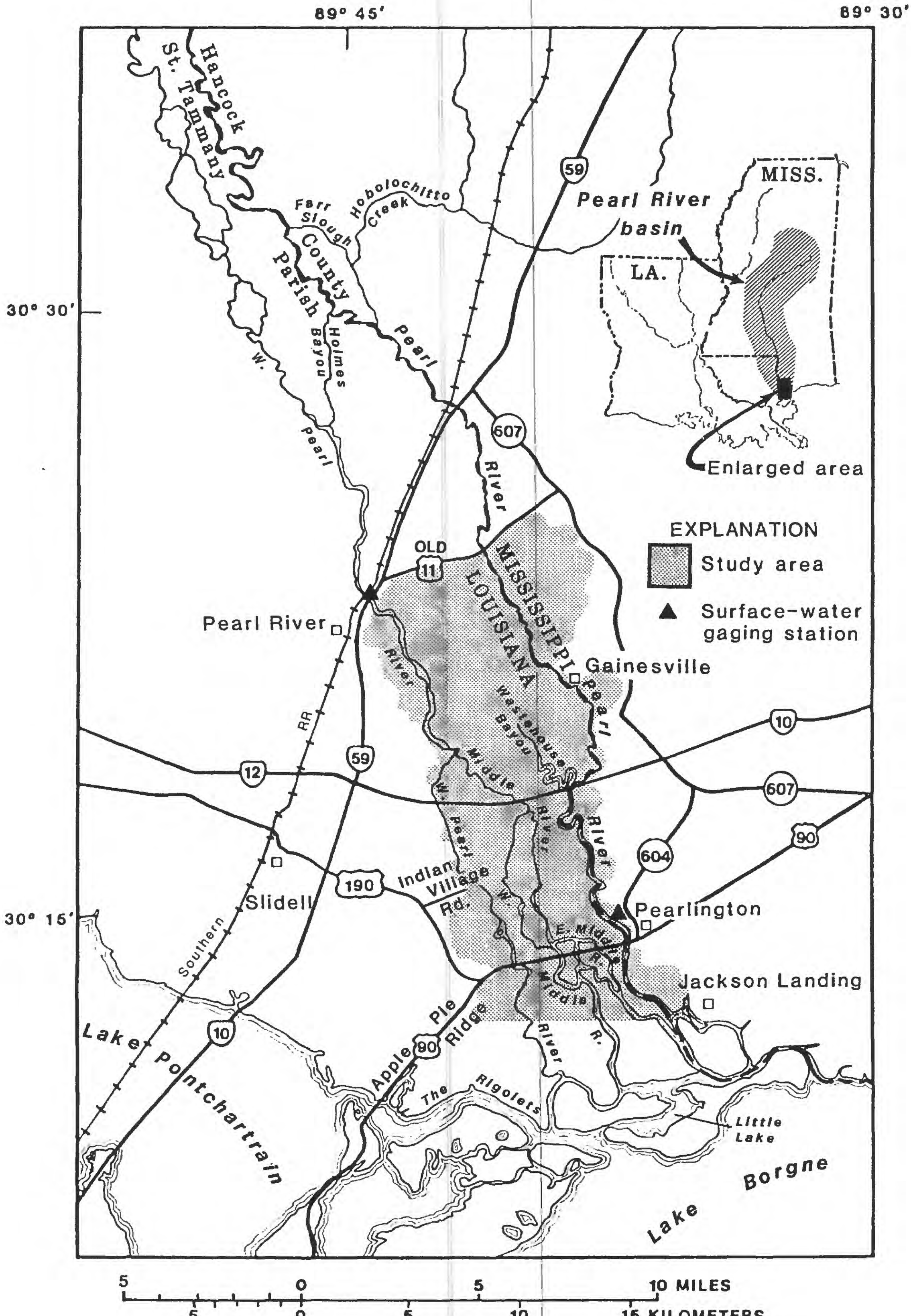

Figure 1.--Lower Pearl River basin and study area, Louisiana and Mississippi.

(U.S. 190). Both the westbound and eastbound lanes of I-10 were overtopped. The eastbound lane was closed for 6 days, and the westbound lane remained closed for an additional 7 days because of structural repair work on the Middle River bridge. U.S. 190 was overtopped and damaged during the 1983 flood although flow over the road at that location in 1980 was insignificant.

Many local residents attribute part of the flooding in the Slidell area to backwater caused by highway embankments that cross the flood plain. In response to this concern, the U.S. Geological Survey, in cooperation with the Louisiana Department of Transportation and Development, Office of Highways, and the U.S. Department of Transportation, Federal Highway Administration, initiated a study to investigate the effect of U.S. 90 on flood elevations.

On highways that cross rivers having wide flood plains, the roadways leading to bridges are generally built above design flood elevations on top of earthen embankments. When approach embankments significantly encroach upon the flood plain, large amounts of backwater may result.

Traditional methods generally used in the hydraulic analysis of multiple-opening bridge crossings on wide flood plains are basically one-dimensional, providing only longitudinal variations in velocity and water-surface elevation. Methods for selecting the distribution of flow through multiple openings in embankments and procedures for determining backwater are based on laboratory investigations and practical experience (Davidian and others, 1962; Bradley, 1970).

Wide flood plains, however, generally exhibit a two-dimensional flow pattern (in the horizontal plane) at high stages. The situation is further complicated by the presence of obstructions, such as highway embankments, and the existence of more than one river channel in the flood plain.

This investigation assessed the effect of U.S. 90 embankments on flooding of the Pearl River near Slidell, La., using the Finite-Element Surface-Water Modeling System for Two-Dimensional Flow in the Horizontal Plane (FESWMS-2DH). This model was used for two reasons: (1) The model can simulate flow with both lateral and longitudinal variations in water-surface elevation and velocity, and (2) the model has been successfully used by other investigators to study flows in similar conditions.

In a previous study of the lower Pearl River at I-10, Lee and others (1983) demonstrated the capability of the modeling system to simulate steady-state flow in a complex, multichannel river flood-plain system having variable topography and vegetative cover. They illustrated the model's capability to simulate lateral and longitudinal variations in velocities and water-surface elevations associated with variable topography and vegetative cover. In addition, the model simulated geometric features such as highway embankments, dikes, and channel bends. Wiche and others (1984) used FESWMS-2DH to simulate the impact of structural and nonstructural modifications to the I-10 embankment on backwater and flow distribution. They simulated (1) the removal of vegetation on overbanks at bridge openings to accelerate flows, (2) construction of two alternate bridge structures, and (3) removal of spoil in control sections within the flood plain.

Purpose and Scope

The objective of this study was to determine the effect of the U.S. 90 embankments on water-surface elevations in the lower Pearl River flood plain during floods of April 1980 and April 1983. The report presents the calibration and verification of the two-dimensional model, FESWMS-2DH, to the lower Pearl River near U.S. 90. Constrictions of the flood plain created by highway embankments, together with other physical features of the flood plain which caused significant lateral variations in water-surface elevation and flow distribution during large floods, were simulated by the model.

Data collected during and after the 1980 flood were used to calibrate the model. The 1983 flood was simulated using the calibrated model with a few minor additions and modifications, and the results were compared with measured values to validate the accuracy of the model.

Acknowledgments

The assistance of Henry Barrouse and William Jack, Louisiana Department of Transportation and Development, Office of Highways; and Mitchell Smith, U.S. Department of Transportation, Federal Highway Administration, is gratefully acknowledged.

Definition of Terms

Throughout this report, the words "right" and "left" refer to positions that would be reported by an observer facing downstream. The terms "backwater" and "drawdown" are used to indicate an increase or decrease, respectively, in water-surface elevation caused by highway embankments. Unit discharge is the discharge per foot of bridge opening. Elevations are referenced to the National Geodetic Vertical Datum of 1929 (NGVD of 1929) defined as a geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada, formerly called mean sea level. Hereafter, NGVD of 1929 is referred to as sea level in this report.

DESCRIPTION OF THE STUDY AREA

The study reach of the Pearl River is located in the lower part of the Pearl River basin along the Mississippi-Louisiana border and is shown in figure 1. The study reach, approximately 14 mi long, is bounded on the north by old U.S. Highway 11 (old U.S. 11) and Interstate Highway 59 (I-59) and ends approximately 2 mi south of U.S. 90. The eastern and western boundaries are the natural bluffs at the edge of the flood plain, where ground-surface elevations rise abruptly to 15 to 25 ft above sea level in the northern part of the study reach and to 5 to 15 ft above sea level in the southern part.

Within the study reach, the axis of the flood plain trends south-southeast, and the flood plain varies in width from about 4 to 7 mi. The flood plain is covered by dense woods, mixed with underbrush in many places. The vegetative cover of the study area is shown in figure 2. Ground-surface

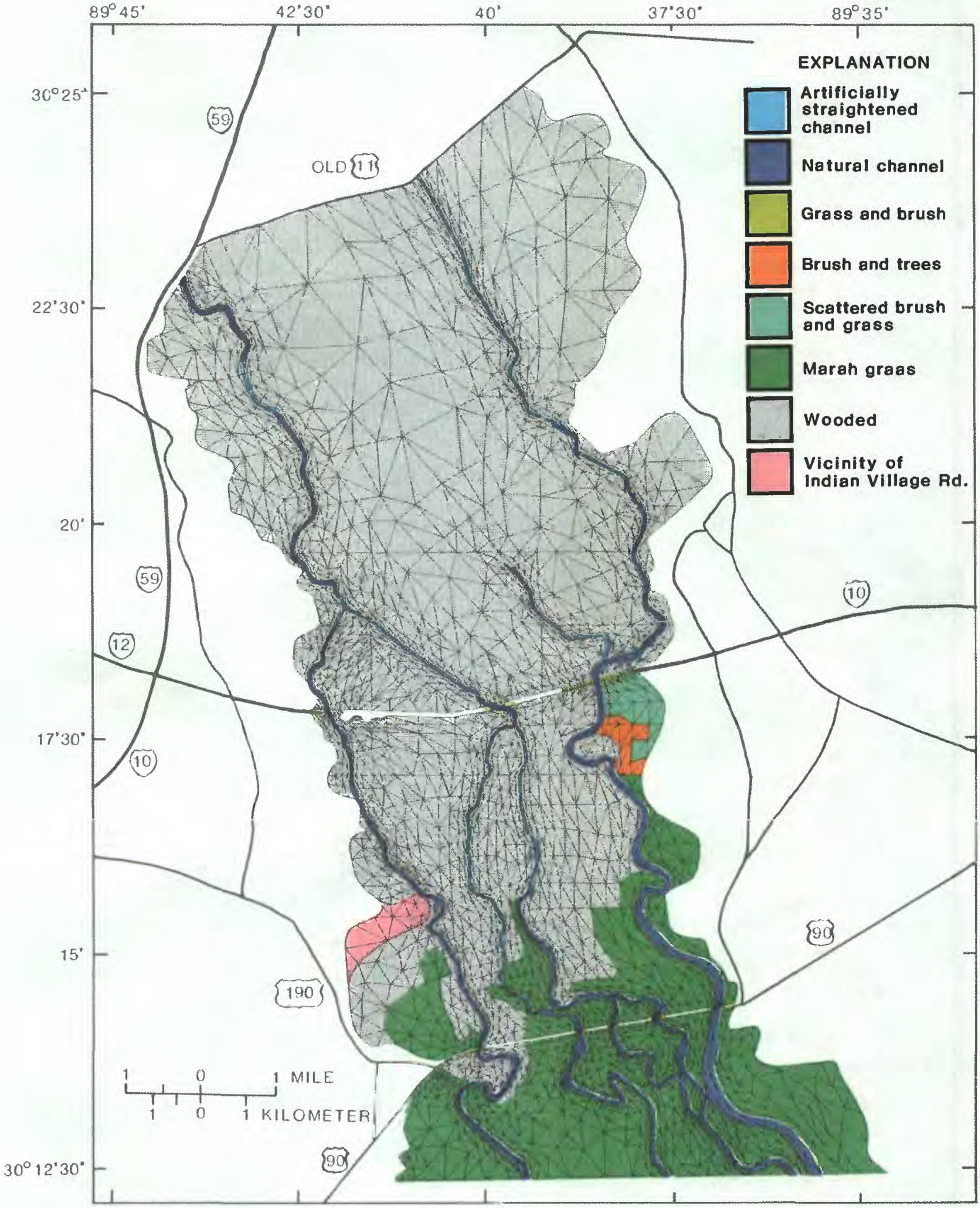

Figure 2.--Grid with roughness types shaded.

elevations of the flood plain range from 0.5 ft above sea level in the southern part of the study area to 15 ft above sea level in the northwestern part. Between the upstream boundary and I-10, ground-surface elevations are higher near the west side of the flood plain than on the east side. Low natural berms border most of the channels in the study reach. The flood plain has a slope of about 1 ft/mi.

The major channels in the study area are the Pearl, East Middle, Middle, West Middle, and West Pearl Rivers, and Wastehouse Bayou. In the northern part of the study reach, the West Pearl River is the largest channel in the flood plain. Near Gainesville, Miss., the channel of the Pearl becomes the largest and remains the largest to the mouths of the river system.

At river mile 15.2 on the West Pearl River, a distributary channel, the Middle River, forms and flows southeastward approximately 3.9 mi, where it divides into the Middle and West Middle Rivers. Approximately 6.3 mi farther south, the Middle River divides again and another distributary channel, the East Middle River, forms. The East Middle and Middle Rivers flow into the Pearl River about 1.3 mi north of Little Lake. Wastehouse Bayou forms within the flood plain and is tributary to the Pearl River just north of I-10.

The main river channels generally flow southward and south-southeastward to the mouths of the Pearl River system. The Pearl River flows into Lake Borgne; the West Middle River and the East Mouth of the West Pearl River flow into Little Lake; and the West Mouth of the West Pearl River flows into The Rigolets (fig. 1). The drainage area of the Pearl River system is 8,670 mi^2 at the mouths of the system (Shell, 1981, p. 232).

Flow enters the study reach through the old U.S. 11 bridge opening at the Pearl River, through the I-59 opening at the West Pearl River, and through numerous small openings in the old U.S. 11 embankments. The I-59 opening at the West Pearl River is 2,630 ft wide, and the old U.S. 11 opening at the Pearl River is 570 ft wide.

The I-10 crossing, about 4.4 mi long, spans the flood plain in an east-west direction in the middle of the study reach. There are bridge openings at the Pearl, Middle, and West Pearl Rivers, with widths of 4,980, 770, and 2,240 ft, respectively. The embankment between the Pearl and Middle Rivers is about 0.8 mi long, and the embankment between the Middle and West Pearl Rivers is about 2.1 mi long. The embankments are about 300 ft wide, and the elevation of the roadway is between 12 and 13 ft above sea level. Natural flood-plain elevations near I-10 range from 1 to 3 ft above sea level.

Flow passes through five openings in the U.S. 90 embankments. The opening widths are 960 ft at the Pearl River, 630 ft at the East Middle River, 580 ft at the Middle River, 580 ft at the West Middle River, and 570 ft at the West Pearl River. The embankment crest elevations range from 6.5 to 9.5 ft above sea level with ground surfaces sloping gradually away from the embankment to the natural flood-plain elevations that range from 0 to 2 ft. Flow leaves the study area approximately 2 mi south of the U.S. 90 crossing, near Jackson Landing on the east bank and near the south edge of Apple Pie Ridge on the west bank.

Flooding in 1980 and 1983 was caused by precipitation over the entire Pearl River basin that ranged from 8.6 to 15.0 in. and 4.7 to 18.3 in., respectively. Water-surface elevations at the gaging station, Pearl River at Pearl River, La. (fig. 1), have ranged from 1.5 to 21.1 ft above sea level during the period of record (October 1899-1985). During the 1980 flood, the maximum water-surface elevation at the Pearl River gage was 19.7 ft, and it was 21.0 ft above sea level during the 1983 flood.

Water-surface elevations have ranged from about 2.0 ft below sea level to about 8.4 ft above sea level (September 10, 1965) during the 23-year period of record (1961-85) at the U.S. Army Corps of Engineers gaging station, Pearl River at Pearlington, Miss. (fig. 1) (Harold Doyle, U.S. Army Corps of Engineers, written commun., 1982, 1984). The maximum water-surface elevation at the gage during the 1980 flood was 5.3 ft above sea level and 6.8 ft during the 1983 flood.

After the 1980 flood, the Geological Survey and the Corps of Engineers carried out a coordinated flood-frequency analysis for eight gaging stations on the Pearl River (V. B. Sauer, U.S. Geological Survey, written commun., 1980). Skew values and historical flood data used in the analysis were mutually agreed upon by both agencies. On the basis of the peak stage and the rating curve developed for the West Pearl River at Pearl River gaging station, the peak discharge for the 1980 flood was 184,000 ft^3/s. The recurrence interval for the 1980 flood at Pearl River was slightly greater than a 50-year flood. Based on the peak stage and the rating curve developed for the West Pearl River at Pearl River station as well as measurements made at I-59, I-10, and U.S. 90, the peak discharge for the 1983 flood was 230,000 ft^3/s. The recurrence interval for the 1983 flood is greater than 200 years. Flood-frequency information for the West Pearl River at Pearl River is shown in figure 3.

Recurrence interval (yr)	2	5	10	25	50	100	200	500
Discharge (ft^3/s)	56,500	87,400	111,000	143,000	169,000	198,000	228,000	272,000

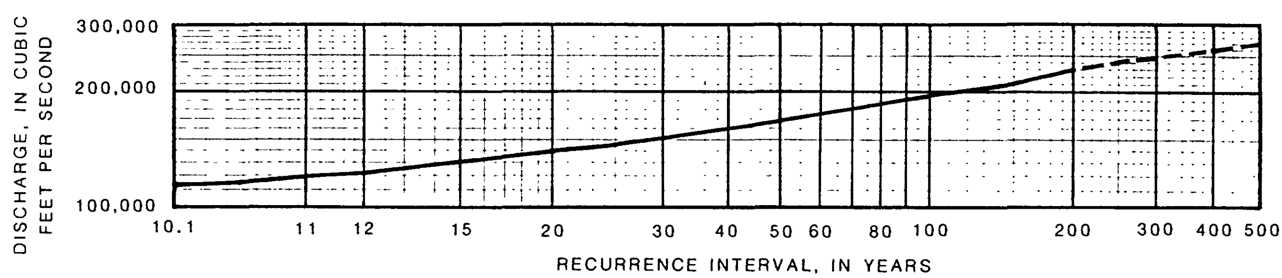

Figure 3.--Flood frequency for West Pearl River at Pearl River, Louisiana.

DESCRIPTION OF MODELING SYSTEM

The FESWMS-2DH is a modular set of computer programs developed specifically for modeling surface-water flows where the flow is essentially two-dimensional in the horizontal plane. The system consists of data preprocessing and postprocessing programs in addition to the central flow model. Preprocessing programs edit and plot input data, and arrange them in appropriate formats for use by 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 the finite-element method of analysis 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 technique are presented.

Governing Equations

A fundamental requirement of any numerical model is a satisfactory quantitative description of the physical processes that are involved. The equations that govern the hydrodynamic behavior of an incompressible fluid are based on the classical concepts of conservation of mass and momentum. For most applications, knowledge of the full three-dimensional flow structure is not required, and it is sufficient to use mean-flow quantities in two dimensions. By integrating the three-dimensional equations over the water depth, and assuming a hydrostatic-pressure distribution and constant fluid density, a set of three equations appropriate for modeling flow in shallow water bodies is obtained. Because the flow is assumed to be in a horizontal direction, it is convenient to use a right-hand Cartesian coordinate system with the x- and y-axes in the horizontal plane and the z-axis directed upwards as shown in figure 4. The x-, y-, and z-components of velocity are denoted by u, v, and w, respectively; z_b is the bed or ground-surface elevation; z_s is the water-surface elevation; and H is the depth of flow.

The depth-averaged continuity equation is

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

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

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

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

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
c_w = wind friction coefficient (dimensionless)
c_f = bottom friction coefficient (dimensionless)
$\hat{\nu}$ = depth-averaged eddy viscosity (foot squared per second)
W = local wind velocity (foot per second)
ψ = angle between the wind direction and the positive x-axis (degrees).

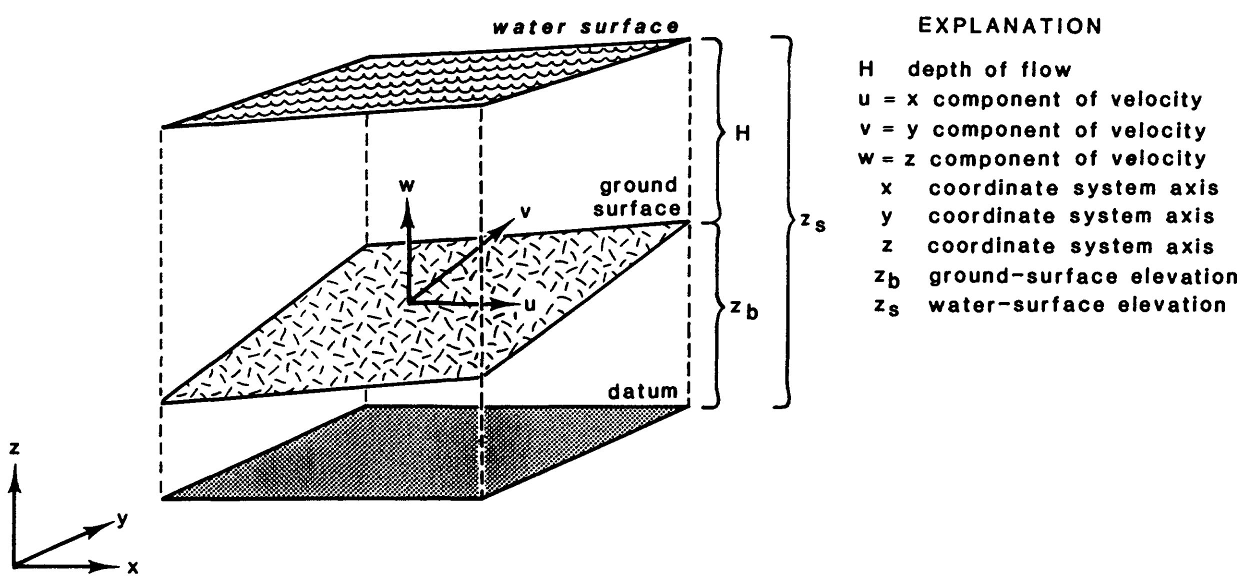

Figure 4.--Coordinate system and symbols.

The bottom friction coefficient can be computed either as

$$c_f = g/C^2 \tag{4}$$

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

$$c_f = gn^2/2.208H^{1/3} \tag{5}$$

where n is the Manning roughness coefficient (second per foot to the one-third power).

The effect of turbulence is modeled using Boussenesq's 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 quantity in the mathematical sense. Rather, this value is defined in such a way that when multiplied by the mean-velocity gradients, the appropriate depth-averaged stress due to turbulence is obtained.

For the simulation of steady-state flow in the study reach of the Pearl River, the time derivative terms in equations 1 through 3 were set to zero. In addition, the Coriolis force due to the Earth's rotation as well as wind friction were deemed negligible and also set to zero. Momentum correction factors (α_{uu}, α_{uv}, and α_{vv}) were all assumed to equal unity.

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 problem, initial conditions must also be specified.

Flow over highway embankments is calculated using the broad-crested weir-flow equation

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

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 between the elevation of the total energy head on the upstream side of the highway embankment and the crest elevation of the embankment.

Solution Technique

The numerical technique used to solve the governing equations is based on the Galerkin finite-element method. In this method, the two-dimensional area being modeled is divided into elements that may be either triangular or quadrangular in shape and can easily be arranged to fit complex boundaries. The elements are defined by a series of node points located at their vertices, midside points, and in the case of nine-node quadrilaterals, at their centers. Values of the dependent variables are then 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, as the equations are usually not satisfied exactly. Weighted averages of the residuals over the entire solution region are computed using numerical integration. Requiring the weighted residuals to vanish allows one to solve for the nodal values of the dependent variables. In Galerkin's method, the weighting functions are chosen to be the same as those used to interpolate values of the dependent variables within each element.

Because the system of hydrodynamic flow equations is nonlinear, Newton's iterative method is used to obtain a solution. In order to apply Newton's method, it is necessary to evaluate not only the governing equations but also a matrix of derivatives with respect to each of the dependent variables for each of the governing equations. This matrix is called the Jacobian, or tangent, matrix and is computed at each iteration in the solution.

MODELING PROCEDURE

The procedure followed in modeling the study reach of the Pearl River included: (1) collection of the necessary topographic and hydraulic data; (2) design of the finite-element network; (3) evaluation of steady-state assumption; (4) assignment of boundary conditions; (5) calibration of the finite-element model on the basis of data collected for the April 1980 flood; (6) validation of the accuracy of the model by simulating the April 1983 flood and comparing the results with measured values; and (7) application of the model to determine the effect of U.S. 90 by simulating the April 1980 and April 1983 floods with the highway embankments removed from the finite-element network. These steps are discussed in more detail in the following sections.

Topographic Data

Topographic data describe the geometry of the physical system under study and include an evaluation of surface roughness to be used in estimating bottom resistance coefficients. Approximately 65 mi of longitudinal channel profiles were obtained for the significant channels in the study reach. Also, 108 representative and special-purpose cross-section surveys were made to define channel geometry (fig. 5). Detailed topographic data at and near bridge openings were obtained from the Louisiana Department of Transportation and Development.

Infrared aerial photographs of the study area and field observations were used to determine vegetation type and density. The collected data were supplemented by historic hydrologic data and U.S. Geological Survey topographic maps.

Hydraulic Data

Hydraulic data consist of measurements of stage and discharge (or stage-discharge hydrographs), high-water marks, rating curves, and limits of flooding. Hydraulic data were used to establish model boundary conditions, and also to calibrate and validate the model.

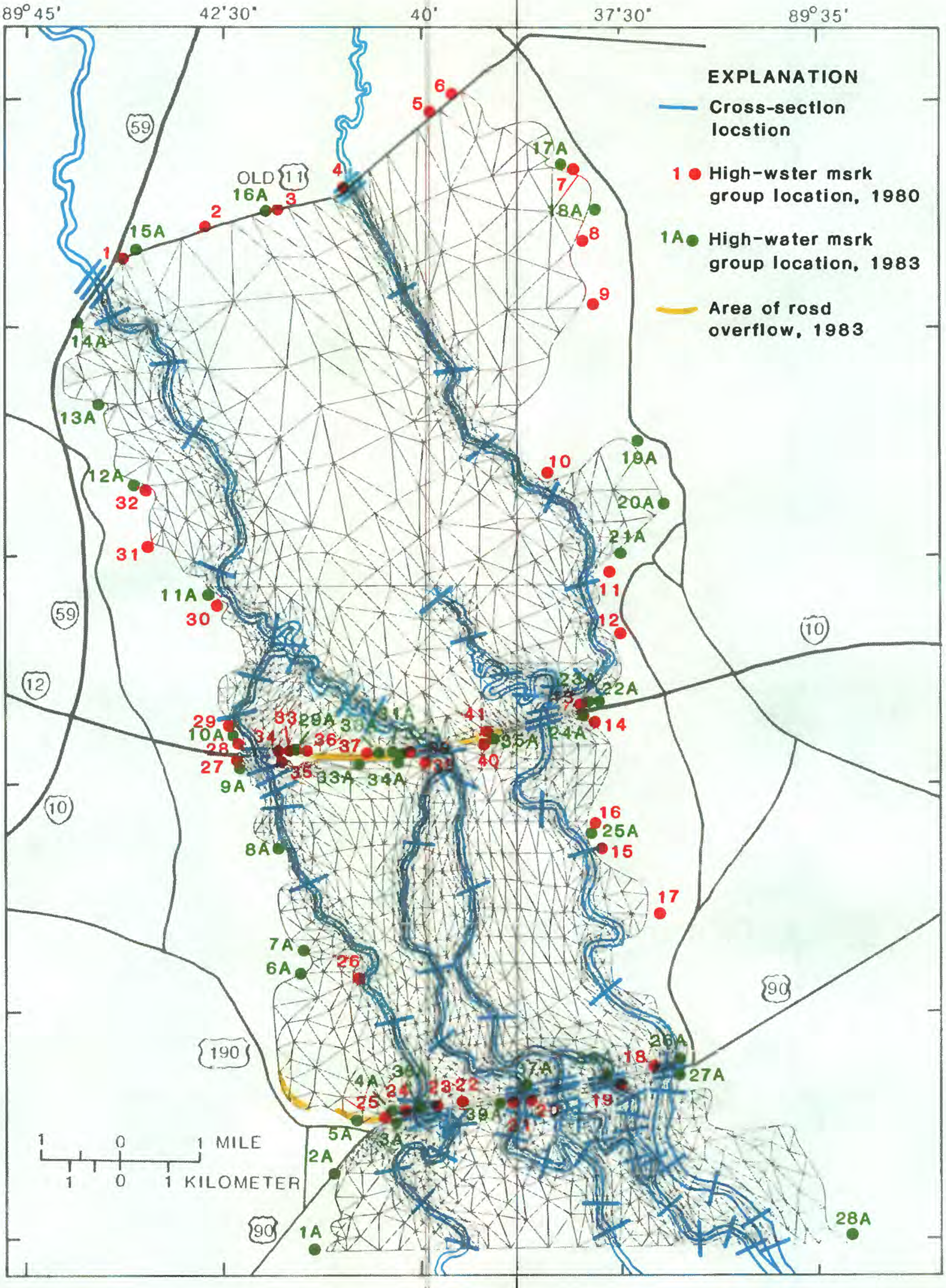

Figure 5.--Location of channel cross sections, high-water mark groups, and areas of road overflow.

Approximately 200 high-water marks within the study area were located by the U.S. Geological Survey during and after both the 1980 and 1983 floods. Because of the close proximity of many of the marks, they were accordingly grouped, and a range of elevations was computed for each grouping. High-water marks for the 1980 flood were grouped at 41 locations and for the 1983 flood at 39 locations. The positions of these high-water mark groupings are shown in figure 5.

A vertical-control net around the study area was established after the 1980 flood. No high-water mark was located more than a mile from the nearest bench mark. Differential leveling from the nearest bench mark to the high-water marks was used to determine the elevations of the high-water marks.

During the 1980 and 1983 floods, discharge measurements were made at or near peak flow at various highway crossings in the study reach. When discharge measurements were not made at the peak, measured values were adjusted using standard techniques to estimate the peak discharge. Each of these discharge measurements along with discharge measurements made during the April 1979 flood are presented in table 1. The April 1979 discharge measurements were used to establish the lateral distribution for the inflow boundary at the upstream end of the study reach.

Design of the Finite-Element Network

Network design may be simply defined as the process of subdividing the area under study into an equivalent system of elements. The basic goal of this subdivision process is to obtain a representation of the study area that will provide an adequate approximation of the true solution at a reasonable cost.

A finite-element network used for a previous study of the 1980 flood (Lee and others, 1983) was modified to simulate both floods in this study. Elements were added to extend the grid approximately 2 mi downstream of U.S. 90 and to simulate flow in the vicinity of Indian Village Road. Extension of the network downstream was necessary to minimize the influence of exit boundary condition specifications on the flow conditions at U.S. 90. (See fig. 2.) Elements near Indian Village Road were added to simulate flow in that area. Although there was some flow in the vicinity of Indian Village Road in 1980, it was considered negligible for modeling purposes. The flow in this vicinity in 1983 was considered significant, requiring additional elements in the finite-element network.

The finite-element network, shown in figure 2, was designed to closely represent the boundary of the area inundated by the 1980 and 1983 floods. The upstream boundary was located parallel to old U.S. 11 and I-59, where inflows could be approximated on the basis of discharge measurements.

After the boundaries were defined, the study area was divided into an equivalent network of triangular elements. Subdivision lines between elements were located where abrupt changes in vegetative cover or topography occur.

Table 1.--Discharges measured during the 1979, 1980, and 1983 floods in the lower Pearl River

[The bridge openings are numbered from left to right as an observer faces downstream]

Discharge, in cubic feet per second

Interstate Highway 59 bridge openings

Date	1 (Pearl River)	2	3	4	5	6	7	8 (West Pearl River)	Total
4-24-79	14,800	2,800	5,500	9,100	4,300	5,100	9,600	91,000	142,000
4-26-79	17,700	3,600	7,400	11,200	5,400	5,800	11,600	92,000	155,000
4-10-83	24,500	6,200	12,100	15,900	10,300	5,000	9,300	112,800	196,000
4-11-83	16,300	3,000	7,500	11,500	4,000	4,900	8,200	89,800	145,000

Interstate Highway 10 bridge openings

Date	Pearl River	Middle River	West Pearl River	Road overflow	Total
4-26-79	88,600	29,000	33,800	------	151,000
5- 1-79	55,000	16,600	18,700	------	90,000
4- 2-80	103,000	30,000	40,800	------	174,000
4-10-83	118,900	30,600	40,100	33,900	223,500

U.S. Highway 90 bridge openings

Date	Pearl River	East Middle River	Middle River	West Middle River	West Pearl River	Total
4-22-80	51,900	11,800	16,700	16,600	6,800	104,000
4-11-83	78,100	23,000	29,200	34,200	23,200	188,000

Each element was designed to represent an area of nearly homogeneous vegetative cover. In areas where velocity, depth, and water-surface gradients were expected to be large, such as near bridge openings and in areas between overbanks and channel bottoms, network detail was increased to facilitate better simulation of the large gradients by the flow model.

The complex geometry of the flood plain of the Pearl River was modeled in detail. Most prototype lengths and widths were realistically represented in the model; however, to reduce the number of elements in the network, several approximations were made. Only large channels were included in the network. Prototype channel cross sections were represented in the model by either triangular or trapezoidal cross sections with areas equal to the measured areas (fig. 6). Some meandering channel reaches having relatively small flows were replaced with artificially straightened, but hydraulically equivalent reaches. (See Lee and others, 1983.) The width of simulated stream channels was kept to a minimum of 200 ft.

The finite-element network contains 7,552 triangular elements and 14,530 computational nodes. In the vicinity of Indian Village Road, 27 elements were not used in the 1980 simulations because flow was considered negligible through the area. Flow was, however, considered significant in 1983 and was simulated by using the 27 elements shown in color (fig. 2).

Evaluation of Steady-State Assumption

Records of gage heights collected at Pearl River, La., at the upper end of the study reach, and at Pearlington, Miss., at the lower end of the reach, were used to evaluate the steady-state assumption for the 1980 flood. Hydrographs for the West Pearl River at Pearl River and the Pearl River at Pearl-

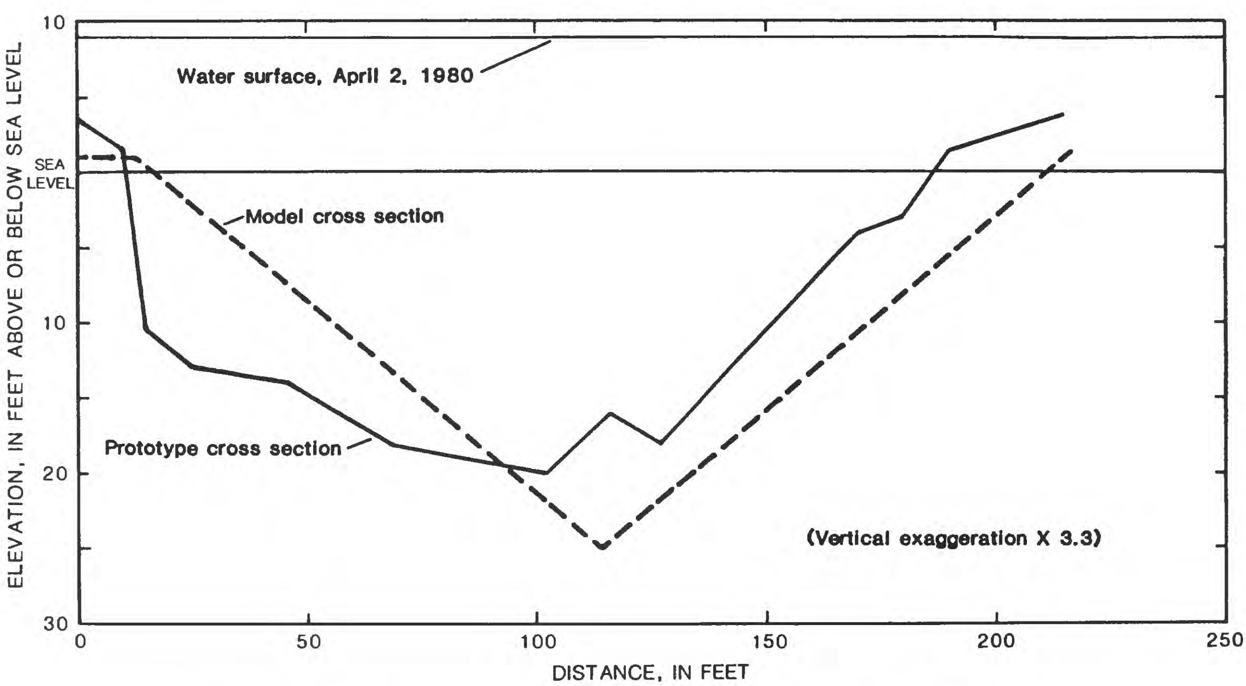

Figure 6.--Prototype and model channel cross sections.

ington for the 1980 flood period are shown in figure 7. At the time of the downstream peak, the upstream water-surface elevation had fallen less than 0.5 ft from its maximum value. On the basis of this observation, it was assumed for modeling purposes that the 1980 flood flow was steady. The steady-state discharge, required as input at the upstream boundary in the model, was obtained from a discharge measurement made by the U.S. Geological Survey at I-10.

Gage heights at Pearlington during the 1983 flood were recorded hourly by local observers. The peak water-surface elevation of 6.77 ft occurred on April 10 at 2200 hours (approximately). Peak water-surface elevation of 21.0 ft occurred at the upstream boundary on April 9 at 2400 hours on the basis of the gage at Pearl River. Hydrographs for the West Pearl River at Pearl River and the Pearl River at Pearlington for the 1983 flood are shown in figure 8. Plots of the water-surface elevations recorded at the Pearl River gage for the 1980 and 1983 peaks show the rate of change in stage for the 1983 flood is much greater than that of the 1980 flood. This shows that the 1983 flood crest was not as steady an event as the 1980 flood. Although the 1983 flood was not as steady as the 1980 event, and the assumption of a steady-state system is used as an approximation, the model produced reasonable results.

Assignment of Boundary Conditions

The peak discharge used as input to the model at the upstream boundary was obtained from discharge measurements and the stage-discharge relation at Pearl River. The discharge of 174,000 ft^3/s at the inflow boundary for 1980 was based on discharge measurements made at I-10 near peak flow. The peak discharge as defined by the stage discharge relation at Pearl River was 170,000 ft^3/s. On the basis of the stage discharge relation at the gaging station at Pearl River, the discharge simulated was 230,000 ft^3/s for the 1983 flood. Several flood discharge measurements made in 1983 which support this peak are used for comparison to computed values at the highway openings. Inflow was concentrated at the old U.S. 11 bridge across the Pearl River and the I-59 bridge across the West Pearl River. Flow into the study reach through numerous small openings in old U.S. 11 was represented as continuous inflow between the east edge of the flood plain and the Pearl River, and between the Pearl and West Pearl Rivers. The distribution of discharge along the upstream boundary of the model is given in table 2 for both the 1980 and the 1983 floods.

Water-surface elevations were specified along the downstream (outflow) boundary for both the 1980 and 1983 flood simulations. For the 1980 flood, high-water marks were not available at the downstream boundary of the model. In order to determine the appropriate values, water-surface elevations at the boundary were varied until computed water-surface elevations along the downstream and upstream sides of the U.S. 90 embankments closely matched the observed high-water marks. For the 1983 flood, specified water-surface elevations were based on high-water marks located on the boundary near Apple Pie

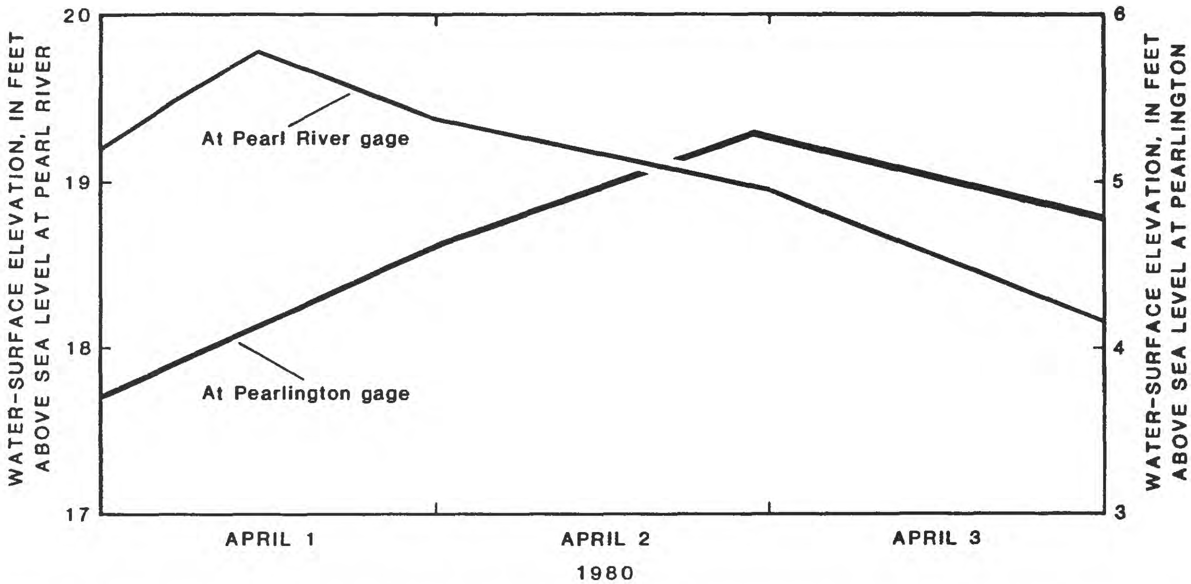

Figure 7.--West Pearl River at Pearl River, Louisiana, and Pearl River at Pearlington, Mississippi, 1980 flood.

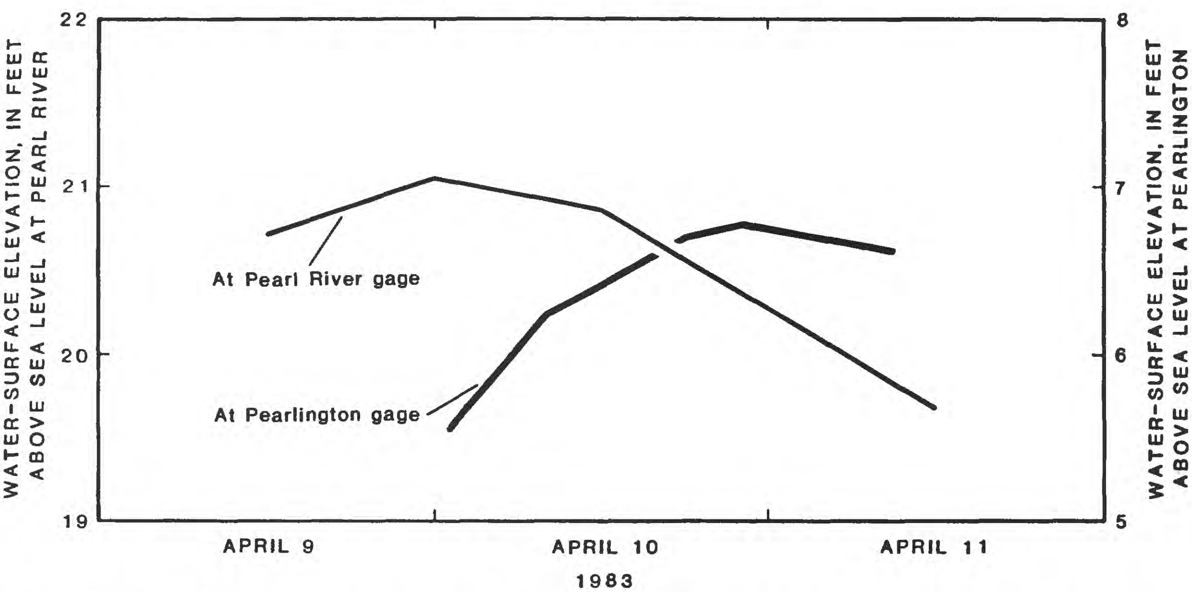

Figure 8.--West Pearl River at Pearl River, Louisiana, and Pearl River at Pearlington, Mississippi, 1983 flood.

Ridge on the west bank of the flood plain and near Jackson Landing on the east bank. The water-surface elevations along the downstream boundary of the model are given in table 3.

Although the discharge of the 1983 flood was greater than the discharge of the 1980 flood, the downstream water-surface elevation specified for the 1983 flood was lower than the water-surface elevation specified for the 1980 flood. Because both tides and wind significantly affect water-surface elevations at the downstream boundary of the model, this seemingly anomalous condition is not considered to be inconsistent with what is observed naturally.

Table 2.--Distribution of discharge at the upstream model boundary

Section of upstream boundary	1980 Discharge, in cubic feet per second	1980 Discharge, as percent of total discharge	1983 Discharge, in cubic feet per second	1983 Discharge, as percent of total discharge
Flood plain between east edge of flood plain and Pearl River.	22,100	12.7	29,200	12.7
Pearl River bridge opening--------	22,000	12.6	29,000	12.6
Flood plain between Pearl and West Pearl Rivers.	32,900	18.9	43,400	18.9
West Pearl River channel----------	69,100	39.6	91,200	39.6
Flood plain between West Pearl River and west edge of flood plain.	28,200	16.2	37,200	16.2
Total--------------------------	174,300	100.0	230,000	100.0

Table 3.--Water-surface elevations at the downstream model boundary

Location[1]	1980	1983
	Water-surface elevation, in feet above sea level	
Jackson Landing (east side)--	4.0	3.2
Apple Pie Ridge (west side)--	4.0	3.4

[1] Water-surface elevation along the downstream boundary between the east and west edges of the flood plain were linearly interpolated.

At all other boundaries, with the exception of areas with weir flow, a zero normal flow condition was specified whereby flow is allowed parallel to the boundary with the requirement of zero net flow across the boundary.

Weir Flow

Weir flow was simulated in locations where significant flow over roadways occurred. The weir length associated with each node along the embankment in areas of roadway overflow was determined from field survey data and topographic maps. Weir crest elevations were based on field survey data. A weir coefficient of 3.0 was used throughout. The value of 3.0 is reasonable to apply for weir computations for a roadway of this type (Hulsing, 1968, p. 27; Bradley, 1970, p. 46).

Along I-10, weir flow was simulated over the roadway as an outflow from the system on the upstream side of the embankment and an inflow to the system on the downstream side of the embankment. Along U.S. 190 north of the intersection with U.S. 90, flow was simulated to exit the study reach. Road overflow in each of these areas was considered significant for modeling purposes only in the 1983 flood; no road overflow was simulated for the 1980 flood.

Flow across roadways was simulated at four locations in the study area for the 1983 flood (fig. 5). Two weir segments were simulated at I-10 and two at U.S. 190. Weir flow is simulated with the model by specifying the nodal location in the network where weir flow is to be computed, weir length to be simulated at that node, a discharge coefficient, and weir crest elevation. Flow is computed for weir segments using the computed water-surface elevation at the specified node number and weir data for that node using equation 6. Weir flow may be specified to re-enter the model network at another node or to leave the system.

The road overflow simulated at I-10 was approximately 15 percent of the total flow and agrees with measured road overflow within 10 percent error. The road overflow simulated at U.S. 190 was only 5 percent of the total flow but compares with measured data with 36 percent error. Weir data are presented in table 4.

Table 4.--Weir specifications

Location	Number of nodes representing weir	Weir crest elevation, in feet above sea level	Weir length, in feet
I-10 embankment between Pearl and Middle Rivers.	10	12.7	4,980
I-10 embankment between Middle River and West Pearl.	20	12.7	8,070
U.S. 190 near U.S. 90 junction-------	3	7.0	930
U.S. 190 approximately 0.5 mile west of U.S. 90 junction.	3	7.2	1,800

Calibration

The mathematical model is a simplified, discrete representation of a complex and continuous physical flow situation. Three-dimensional topographic features are represented by elements, and the physics of flow are assumed to obey differential equations in which certain empirical coefficients appear. Calibration is the process of adjusting the values of empirical coefficients, ground-surface elevations, and the number, size, and shapes of elements defining the finite-element network so that the flow simulated by the model will reproduce as closely as possible the comparable natural events. Calibration is not the process of force-fitting computed results to observed data. Empirical coefficients must be assigned that are reasonably consistent with what would be expected based on engineering judgement. While the two-dimensional elements defining the topography of the study reach may not directly include every feature of the area, those features that do significantly affect the flow must be modeled in appropriate detail.

The finite-element flow model was calibrated using data collected during and after the April 1980 flood. Calibration consisted of matching as closely as possible all observed high-water mark groupings and measured discharges at the three bridge openings in I-10. Minor adjustments to the initially assigned Chézy coefficients gave close agreement between simulated and observed data in most instances. In making the initial estimates and subsequent adjustments, care was taken to insure that the Chézy values were physically reasonable and consistent throughout the study reach. Areas to which different Chézy coefficients were assigned are shown in figure 2. Descriptions of these areas and the assigned values are given in table 5.

On the basis of previous simulations, the value of depth-averaged eddy viscosity, $\hat{\nu}$, was set to 100 ft^2/s for all elements in the network. Numerical experiments indicated that once the value of the eddy viscosity was set high enough to ensure convergence, the general solution was much less sensitive to changes in this coefficient than to changes in the values of Chézy coefficients. Because of a lack of information about their correct values, and to avoid convergence problems, the value of the depth-averaged eddy viscosity was maintained at 100 ft^2/s for all elements in the network.

The computed water-surface elevation at the 41 high-water marks (fig. 5) agree with the observed value within ± 0.2 ft at all but seven locations, where they are within ± 0.6 ft (table 6). The root-mean-square difference between the computed and observed values is 0.2 ft, the average absolute error is 0.1 ft. Water-surface elevation contours for the simulation of the 1980 flood are shown in figure 9. Water-surface elevation profiles along the east and west edges of the flood plain for the computed and observed values are shown in figure 10.

Discharge measurements made at I-10 in 1980 were, for all practical purposes, made at peak flow; and although I-10 was overtopped, the amount of road overflow was negligible. The computed and observed discharges through each opening compare favorably with a root-mean-square error of 4.8 percent. The largest difference of 5.1 percent occurs at the Middle River opening. Unit discharge is the discharge per foot of bridge opening. Comparisons of the computed and observed unit discharge across the bridge openings are shown in figures 11, 12, and 13.

Verification

Verification involves the testing of a calibrated model to see if the simulated values compare favorably with field data from an event that is independent of the event used for calibration. If a model satisfactorily reproduces these data without any further parameter adjustments, the model is then able to be applied to conditions outside the range of calibration with a significant degree of confidence.

Table 5.--Values of Chézy coefficients

Element description or location	Chézy coefficient (feet$^{1/2}$ per second)
Flood plain	
Woods	20
Low marsh grass in southern part of study reach	32
High marsh grass in southern part of study reach	25
Marsh grass and brush downstream from I-10 bridge across Pearl River.	27
Brush and trees south of preceding marsh-grass area	19
Pearl River	
Natural channel between river miles 6.0 and 9.0	105
Natural channel between river miles 9.0 and 15.9	95
Natural channel between river miles 15.9 and 19.2	77
Straightened channel between river miles 19.2 and 20.3	77
Natural channel between river miles 20.3 and 20.9	77
Straightened channel between river miles 20.9 and 26.3	77
Wastehouse Bayou	
Straightened channel between river miles 0.0 and 4.4	54
East Middle River	
Natural channel between river miles 1.8 and 2.7	77
Middle River	
Natural channel between river miles 2.3 and 5.4	77
Straightened channel between river miles 5.4 and 9.0	60
Natural channel between river miles 9.0 and 10.0	77
Straightened channel between river miles 10.0 and 12.9	62
West Middle River	
Natural channel between river miles 5.9 and 8.0	77
Straightened channel between river miles 8.0 and 12.7	68

Table 5.--Values of Chézy coefficients--Continued

Element description or location	Chézy coefficient (feet$^{\frac{1}{2}}$ per second)
West Pearl River	
Natural channel between river miles 7.9 and 14.9---------	77
Straightened channel between river miles 14.9 and 15.9---	46
Natural channel between river miles 15.9 and 19.4--------	91
Straightened channel between river miles 19.4 and 20.4---	85
Natural channel between river miles 20.4 and 21.4--------	91
Natural channel between river miles 21.4 and 21.9--------	105
Old Pearl River	
Natural channel between river miles 0.0 and 2.5----------	77

Data collected for the April 1983 flood were used to verify the flow model. Overtopping during the 1980 flood was not considered significant; however, flow over the roadways in 1983 was considered significant. Flows over the roadways were computed as weir flow in the model, thus, requiring the specification of additional information. Both I-10 and U.S. 190 were divided into weir segments for each of which a length, crest elevation, and discharge coefficient were provided. Lengths and crest elevations were accurately determined from topographic maps and field surveys. A base discharge coefficient of 3.0 was assigned to each weir segment.

Computed and observed water-surface elevations at the 39 locations (fig. 5) where high-water marks were grouped agree within ± 0.3 ft at all but seven locations, where they agree within ± 0.8 ft (table 7). The root-mean-square difference between the computed and observed values is 0.28 ft. Water-surface elevation contours for the simulation of the 1983 flood are shown in figure 14. Computed and observed water-surface elevation profiles along the east and west edges of the flood plain are shown in figure 15.

The computed and observed discharge through each opening at the I-10 and U.S. 90 crossings, and over U.S. 190 are listed in table 8. It should be noted that the discharge measurements were made near but not at the peak and were adjusted proportionally to obtain the peak discharge. Good agreement between computed and observed discharges through each bridge opening in I-10 and U.S. 90 is seen. Error at I-10 is 4.9 percent root mean square among the bridges and 6.1 percent root mean square considering bridge openings and road overflow. The absolute mean error is 4.2 percent among the bridge openings and 5.4 percent considering bridge openings and road overflow. Comparisons of computed and observed unit discharge at the bridge openings in U.S. 90 are

Table 6.--Elevations of the computed water surface and observed high-water marks for the 1980 flood

[Average error = 0.1; root-mean-square error = 0.2; datum is sea level]

Location reference number (fig. 5)	Computed water-surface elevation (feet)	Observed high-water mark elevation (feet)	Computed minus observed (foot)
1	18.6	17.9-18.0	0.6
2	17.4	17.2	.2
3	16.8	16.7	.1
4	16.6	16.4	.2
5	16.0	15.6	.4
6	15.9	15.4	.5
7	15.5	15.7-15.8	-.2
8	15.4	15.2	.2
9	15.3	15.0-15.1	.2
10	13.4	13.5	-.1
11	13.0	13.1-13.2	-.1
12	12.1	12.2-12.3	-.1
13	11.9	11.8-11.9	.0
14	10.7	10.8-10.9	-.1
15	9.1	8.9	.2
16	9.3	9.4	-.1
17	8.6	8.6- 8.7	.0
18	6.0	6.2	-.2
19	5.7	5.7	.0
20	5.7	5.8	-.1
21	5.6	5.8	-.2
22	7.1	7.1	.0
23	7.1	6.9	.2
24	7.4	7.3	.1
25	7.6	7.4	.2
26	8.1	8.2	-.1
27	11.6	11.1-11.2	.4
28	11.6	11.6-11.9	.0
29	12.4	12.4-12.7	.0
30	14.1	14.1-14.3	.0
31	14.9	14.7-14.8	.1
32	15.9	15.2-15.6	.3
33	11.6	11.5-11.6	.0
34	11.6	11.7	-.1
35	10.2	10.1-10.5	.0
36	12.5	12.5-12.8	.0
37	12.6	12.7-12.8	-.1
38	9.9	10.3	-.4
39	12.4	12.6-12.7	-.2
40	10.4	10 8-10.9	-.4
41	12.0	11.5-11.8	.2

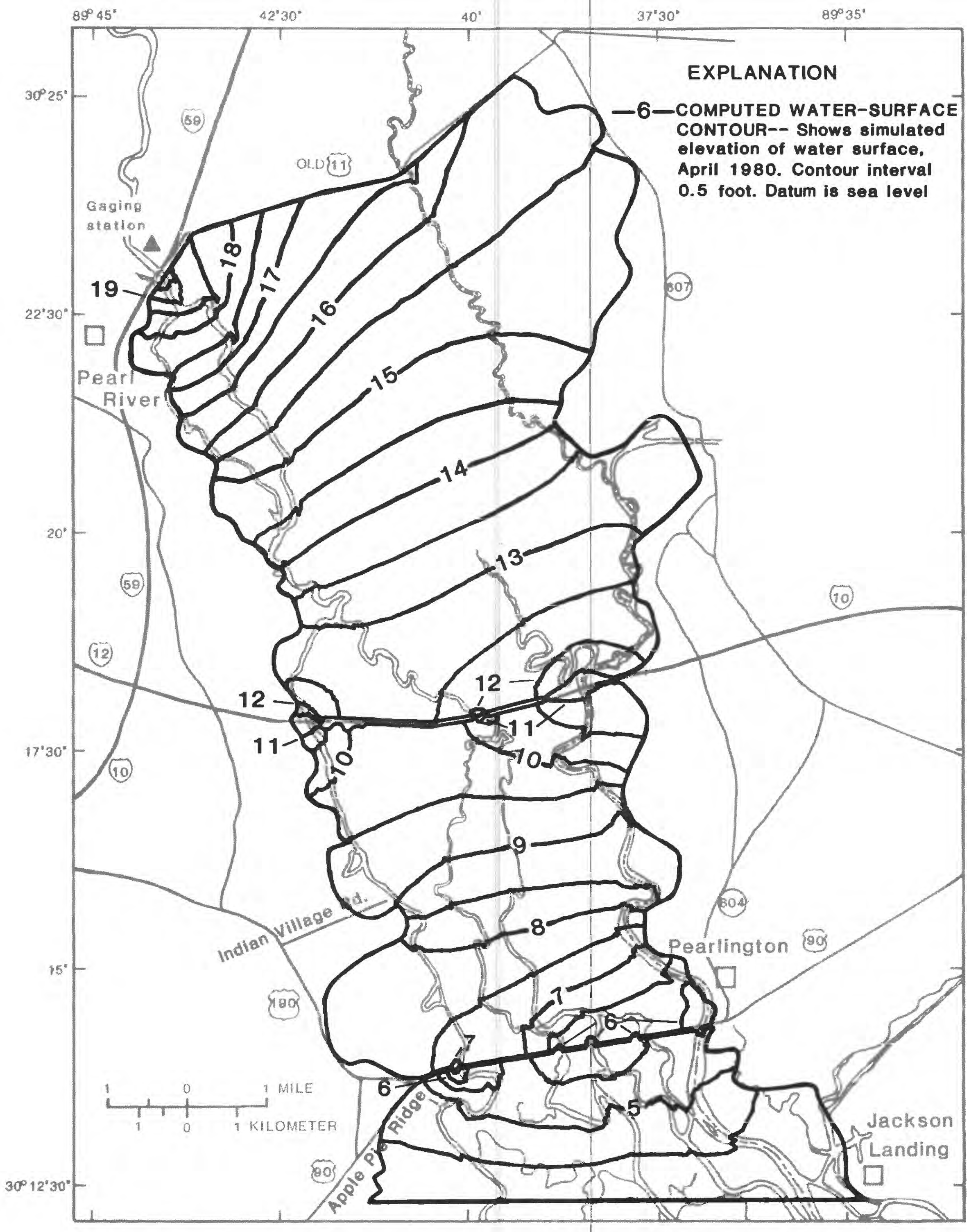

Figure 9.--Water-surface elevation contours, 1980 flood, with the U.S. Highway 90 embankment in place.

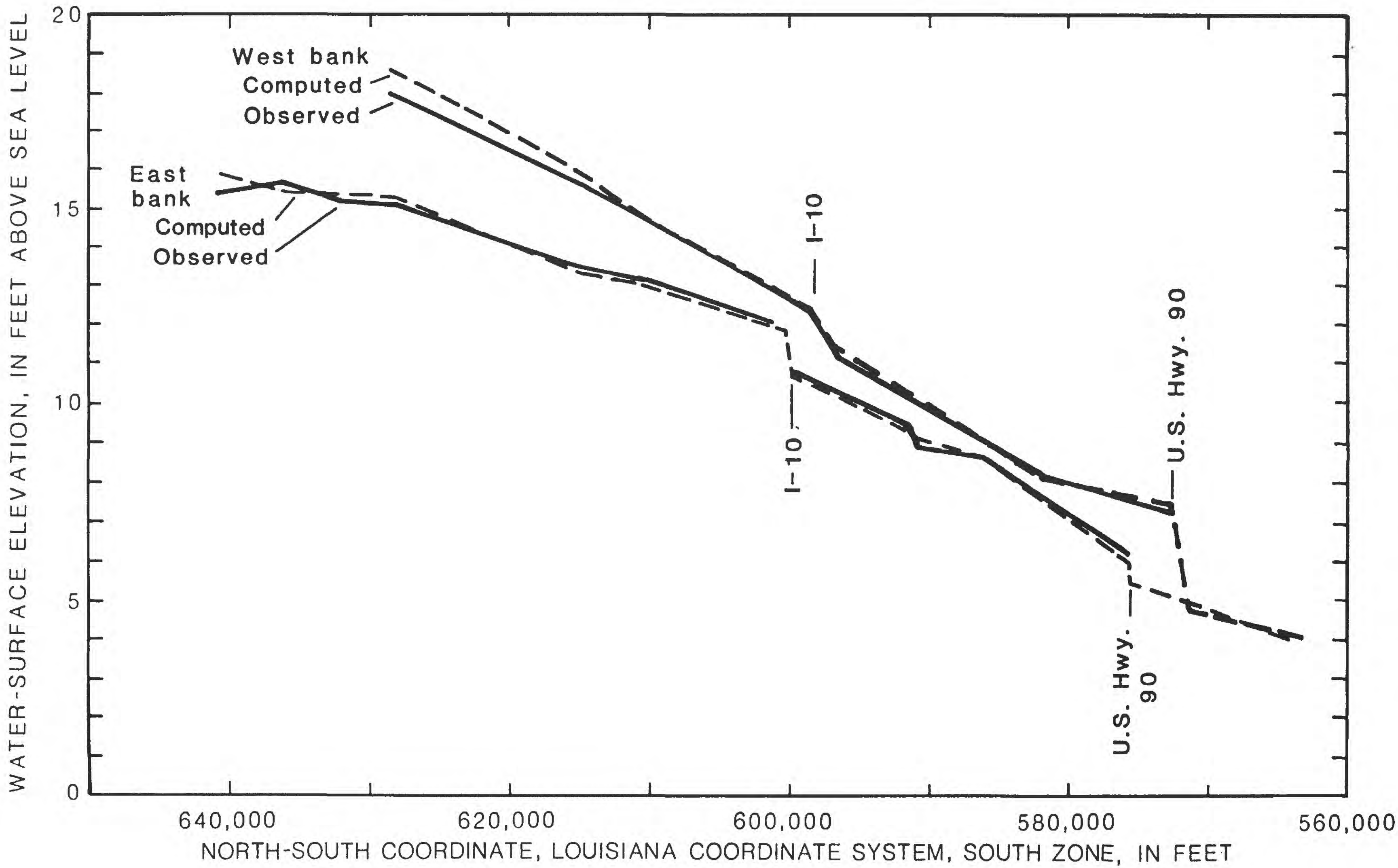

Figure 10.--Computed versus observed water-surface profiles of east and west banks of the flood plain, 1980.

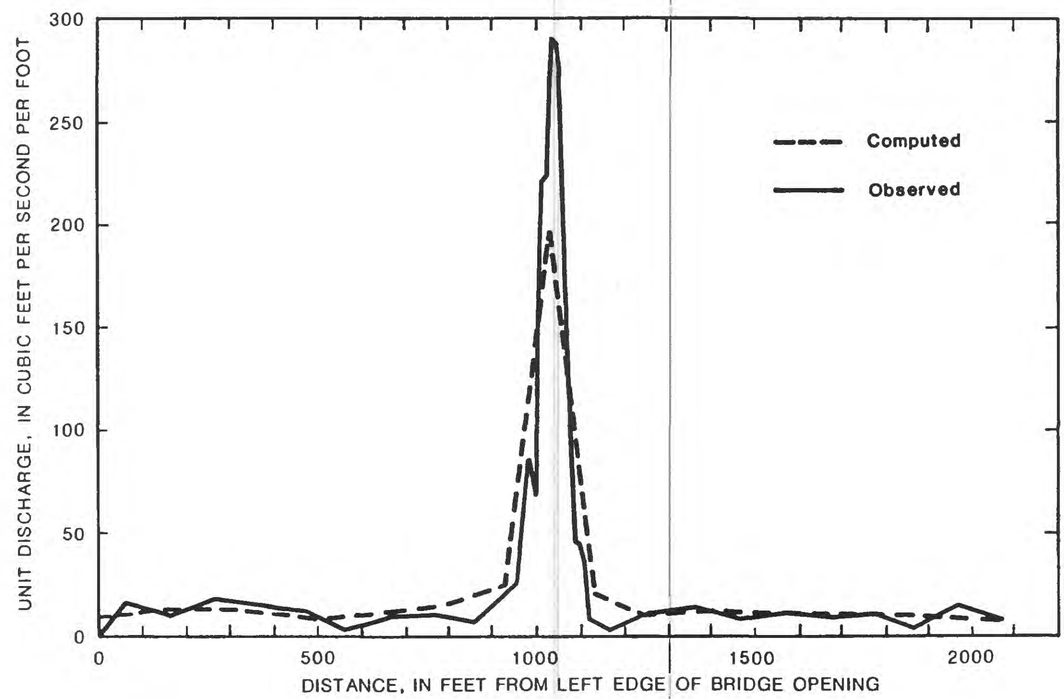

Figure 11.--Computed versus observed unit discharge of the West Pearl River at Interstate Highway 10, 1980 flood.

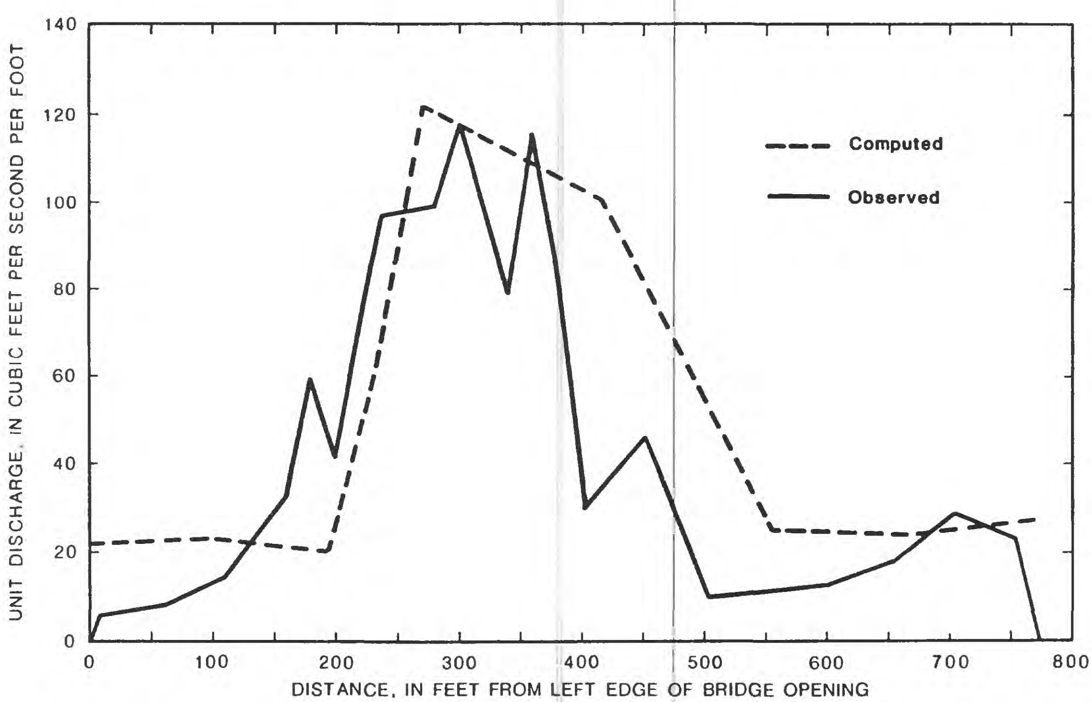

Figure 12.--Computed versus observed unit discharge of the Middle Pearl River at Interstate Highway 10, 1980 flood.

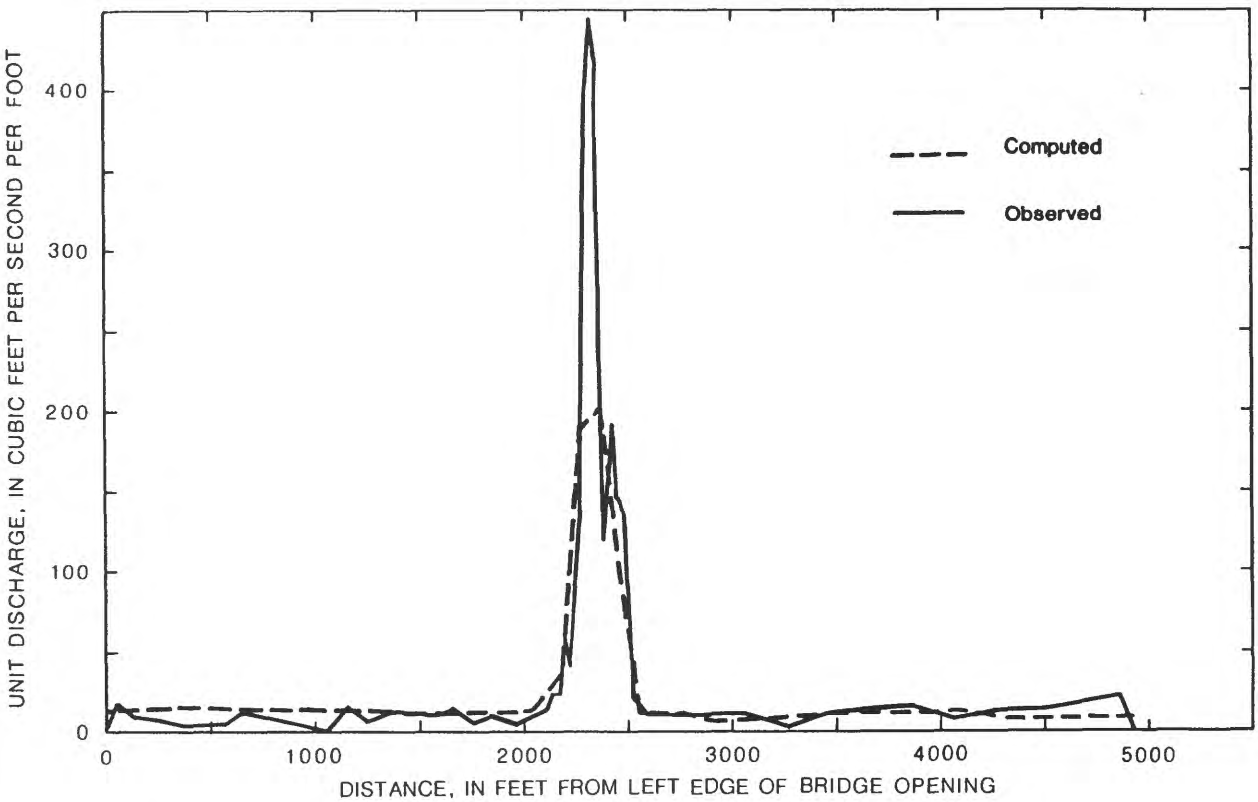

Figure 13.--Computed versus observed unit discharge of the Pearl River at Interstate Highway 10, 1980 flood.

shown in figures 16 through 20, and at the openings in I-10 in figures 21 through 23. Good agreement between computed and observed road overflows on I-10 was obtained even in the absence of calibration of weir flow equation variables. The error at U.S. 90 is 10.5 percent root mean square among the bridges and 17.4 percent root mean square considering bridge opening and road overflow. The largest error at a bridge opening is 17.3 percent ocurring at the West Middle River. The mean absolute error is 9.3 percent among the bridge openings and 13.7 percent considering bridge openings and road overflow. The weir flow simulated over U.S. 190 did not compare to observed data as well as the weir flow at I-10. Flow over U.S. 190, simulated as flow exiting the study area at the model boundary, was 36 percent greater than the measured discharge; however, this flow is only 6 percent of the total flow. The area of approach to the road overflow section was not represented in detail. The possibility exists that there are flow controls upstream of the roadway that were not properly defined in the model, resulting in computational errors in the weir flow. because of the favorable comparison of

Table 7.--Elevations of the computed water surface and observed high-water marks for the 1983 flood

[Average error = 0.2; root-mean-square error = 0.3; datum is sea level]

Location reference number (fig. 5)	Computed water-surface elevation (feet)	Observed high water mark elevation (feet)	Computed minus observed (foot)
1A	3.3	3.4	-0.1
2A	5.0	5.1	-.1
3A	7.1	6.8	.3
4A	8.5	8.2	.3
5A	8.6	8.3	.3
6A	9.6	9.4-10.0	.0
7A	10.0	9.9	-.1
8A	11.2	10.5-10.7	.5
9A	12.8	12.9-13.1	-.1
10A	13.1	13.2	-.1
11A	15.7	15.4	.3
12A	17.3	16.7	.6
13A	18.4	17.8	.6
14A	20.0	19.8	.2
15A	19.8	19.6	.2
16A	18.7	18.4	.3
17A	17.4	17.4-17.5	.0
18A	17.4	16.9	.5
19A	14.6	14.6	.0
20A	14.7	14.6	.1
21A	14.6	14.4	.2
22A	13.4	13.2	.2
23A	13.2	13.2-13.3	-.0
24A	12.1	12.2	-.1
25A	10.5	10.6	-.1
26A	6.2	6.2	.0
27A	5.8	5.6- 5.7	.1
28A	3.1	3.2	-.1
29A	13.6	13.5	.1
30A	13.7	13.7-13.8	.0
31A	13.7	13.6	.1
32A	13.6	13.6	.0
33A	11.4	11.4	.0
34A	11.4	11.8	-.4
35A	11.9	12.5	-.6
36A	7.9	7.1	.8
37A	6.5	6.2	.3
38A	6.7	6.4	.3
39A	6.1	5.9- 6.1	.0

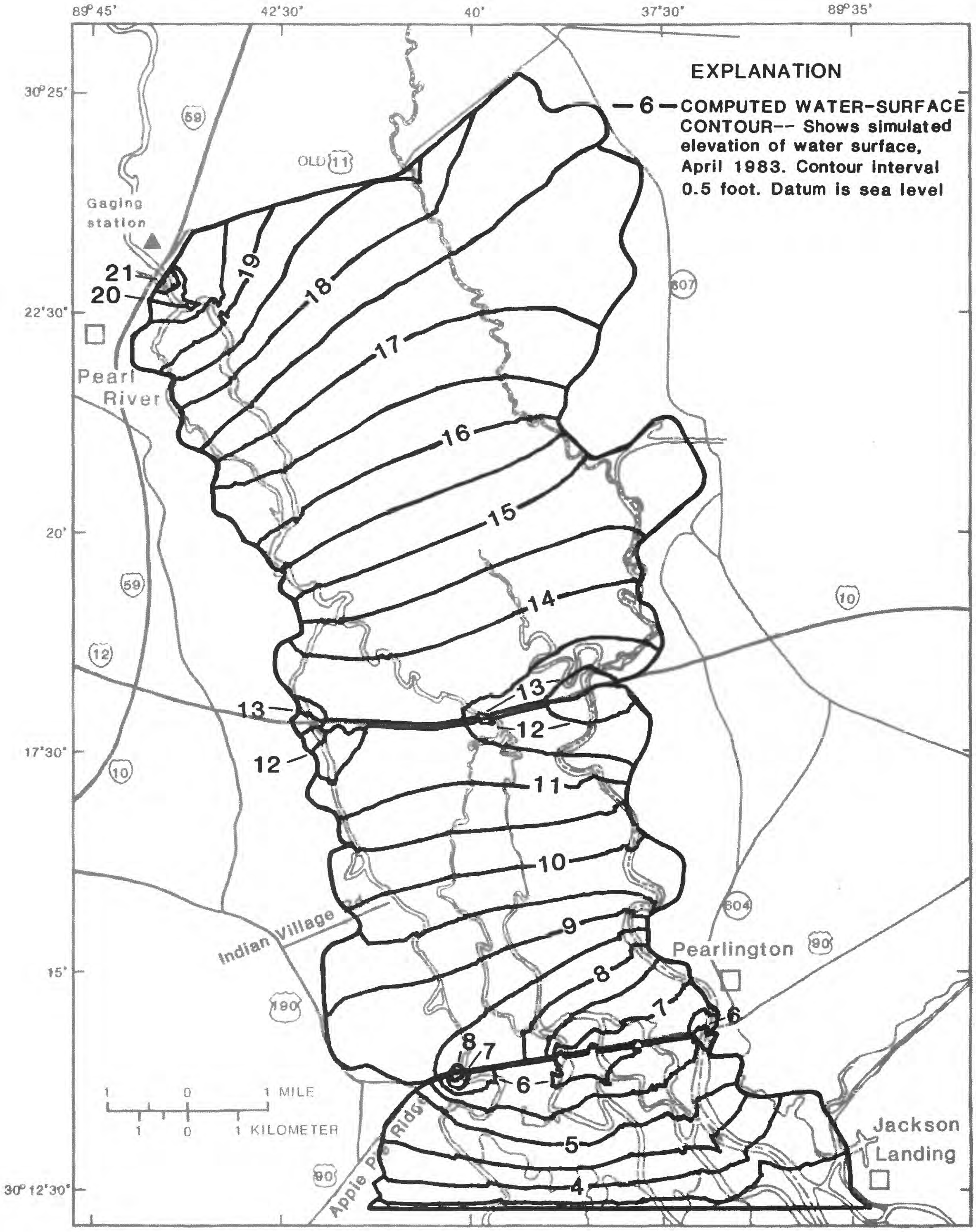

Figure 14.--Water-surface elevation contours, 1983 flood, with the U.S. Highway 90 embankment in place.

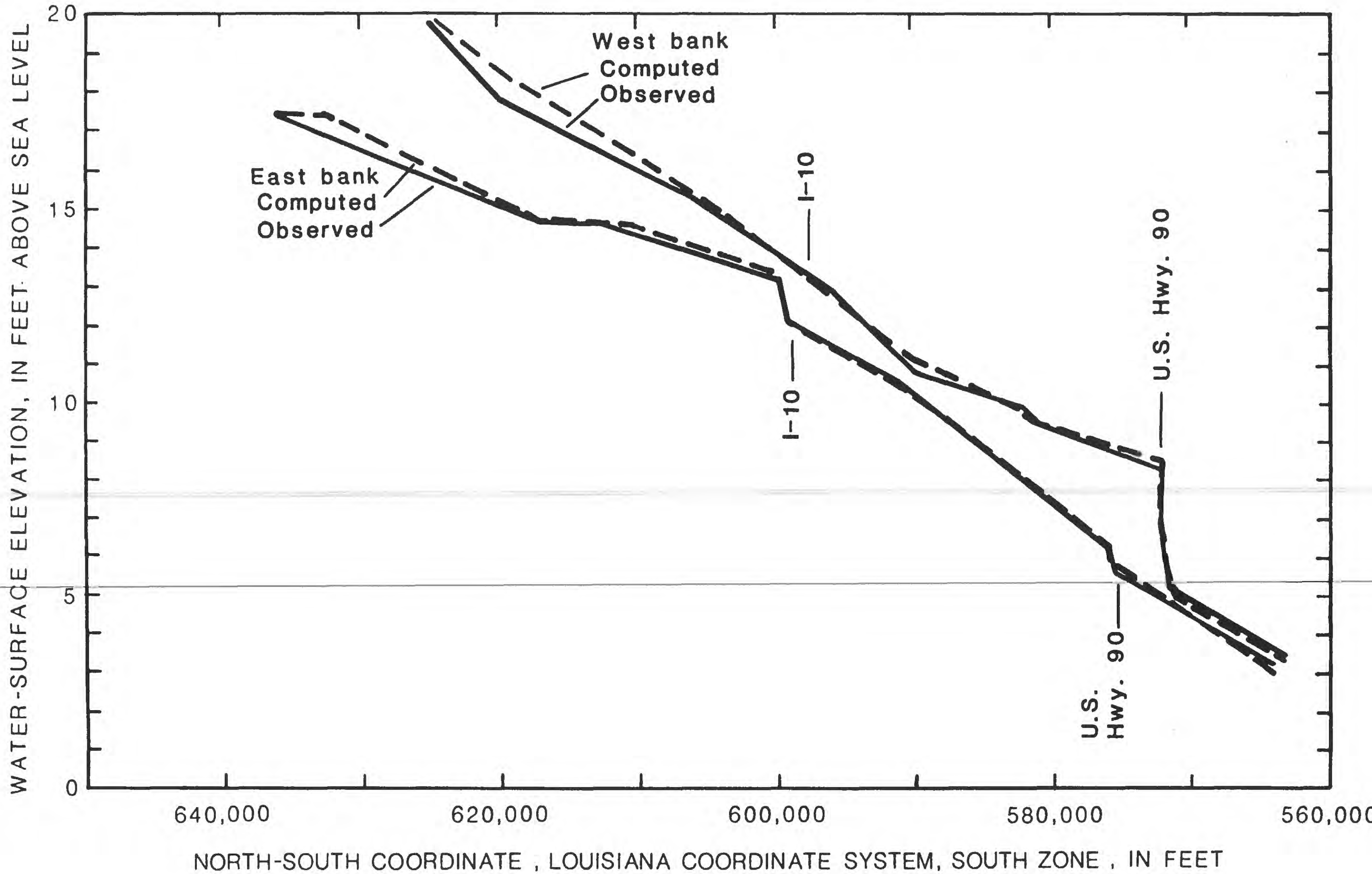

Figure 15.--Computed versus observed water-surface profiles along east and west banks of the flood plain, 1983.

Table 8.--Measured and computed discharges at U.S. Highway 90, Interstate Highway 10, and U.S. Highway 190, 1980 and 1983 floods

[ft³/s, cubic feet per second]

Openings or road overflow	Measured discharge (ft³/s)	Percent of total discharge	Adjusted to peak[1] (ft³/s)	Computed discharge (ft³/s)	Percent of total discharge	Percent error[2]
1980 flood at U.S. Highway 90						
West Pearl River---	-------	---	-------	22,900	13	-----
West Pearl River---	-------	---	-------	36,300	20	-----
Middle River-------	-------	---	-------	27,100	15	-----
East Middle River--	-------	---	-------	19,900	12	-----
Pearl River--------	-------	---	-------	72,700	40	-----
Total------------				178,900	100	
1980 flood at Interstate Highway 10						
West Pearl River---	40,800	22	40,800	39,200	22	-4.5
Middle River-------	30,000	16	30,000	28,500	17	-5.1
Pearl River--------	103,000	62	103,000	104,600	61	-4.9
Total------------	174,000	100	174,000	172,300	100	
Root mean square = 4.8						
1983 flood at U.S. Highway 90						
West Pearl River---	23,200	12	27,100	29,900	12	+10.3
West Middle River--	34,200	17	40,000	46,900	19	+17.3
Middle River-------	29,200	15	34,200	35,700	15	+4.4
East Middle River--	23,000	12	26,900	25,900	11	-3.7
Pearl River--------	78,100	40	91,400	81,600	37	-10.7
U.S. 190 overflow--	8,900	4	10,400	14,100	6	35.6
Total------------	196,600	100	230,000	234,100	100	
Root mean square (openings only) = 10.5						
Root mean square (openings and road overflow) = 17.4						
1983 flood at Interstate Highway 10						
West Pearl River---	40,100	18	41,200	44,300	19	+7.5
Middle River-------	30,600	14	31,500	32,600	14	+3.5
Pearl River--------	118,900	53	122,400	124,200	53	+1.5
Road overflow------	33,900	15	34,900	31,800	14	-8.9
Total------------	223,500	100	230,000	232,900	100	
Root mean square (openings only) = 4.9						
Root mean square (openings and road overflow) = 6.1						

[1] Adjusted to peak = measured X [peak (total)/measured (total)].
[2] Percent error = (Computed - measured)/measured.

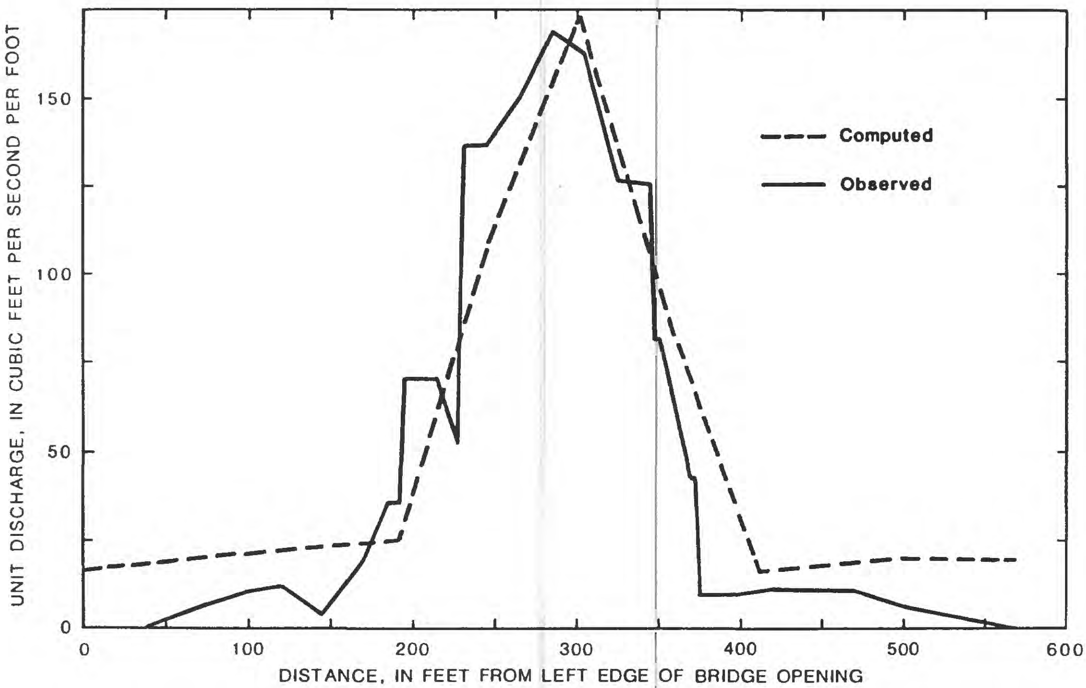

Figure 16.--Computed versus observed unit discharge of the West Pearl River at U.S. Highway 90, 1983 flood.

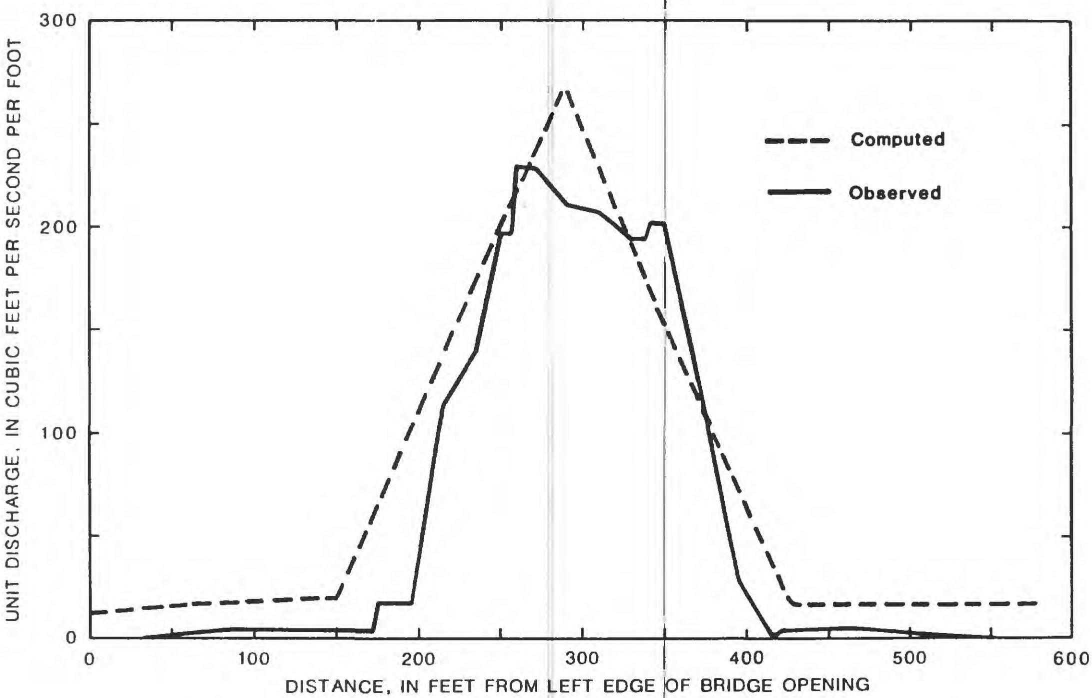

Figure 17.--Computed versus observed unit discharge of the West Middle Pearl River at U.S. Highway 90, 1983 flood.

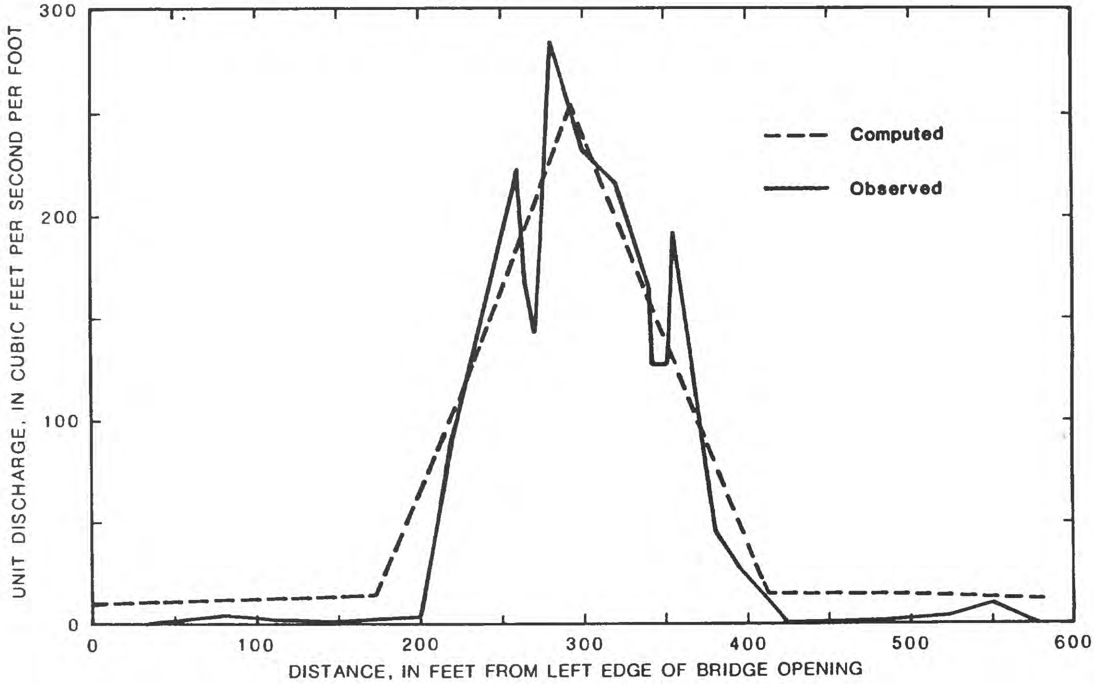

Figure 18.--Computed versus observed unit discharge of the Middle Pearl River at U.S. Highway 90, 1983 flood.

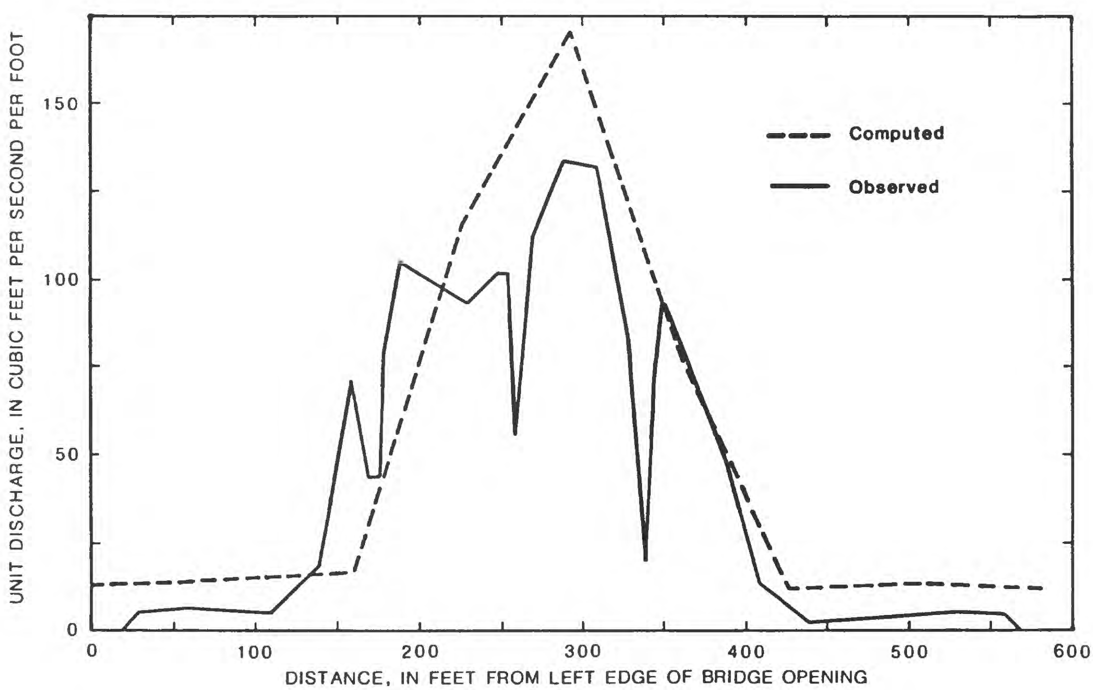

Figure 19.--Computed versus observed unit discharge of the East Middle Pearl River at U.S. Highway 90, 1983 flood.

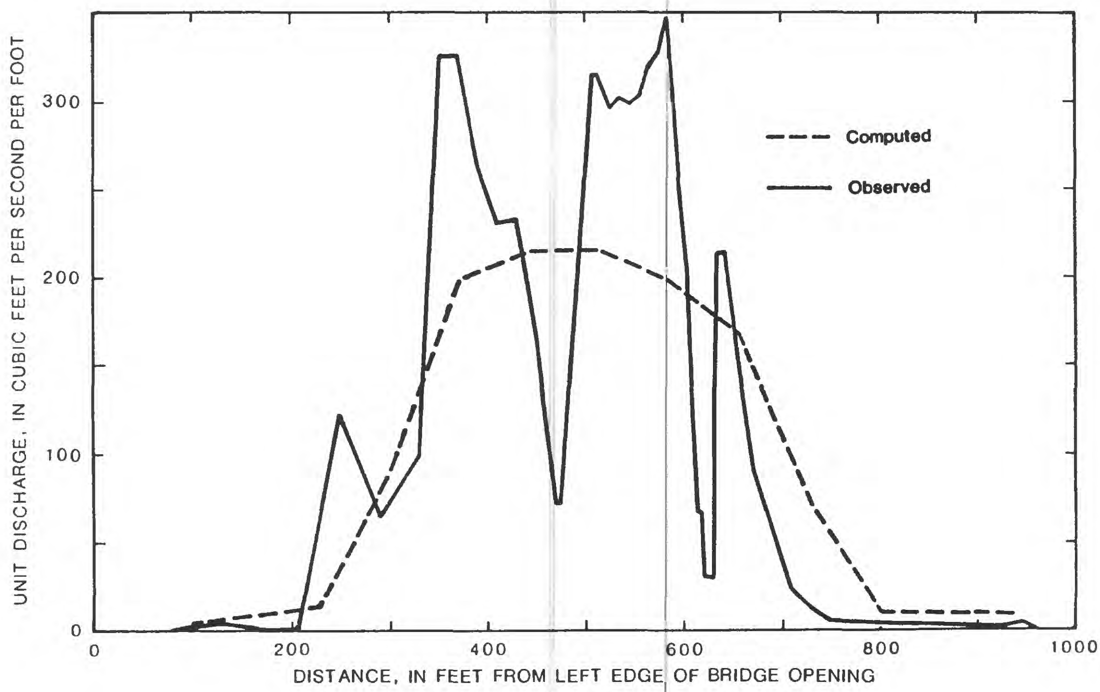

Figure 20.--Computed versus observed unit discharge of the Pearl River at U.S. Highway 90, 1983 flood.

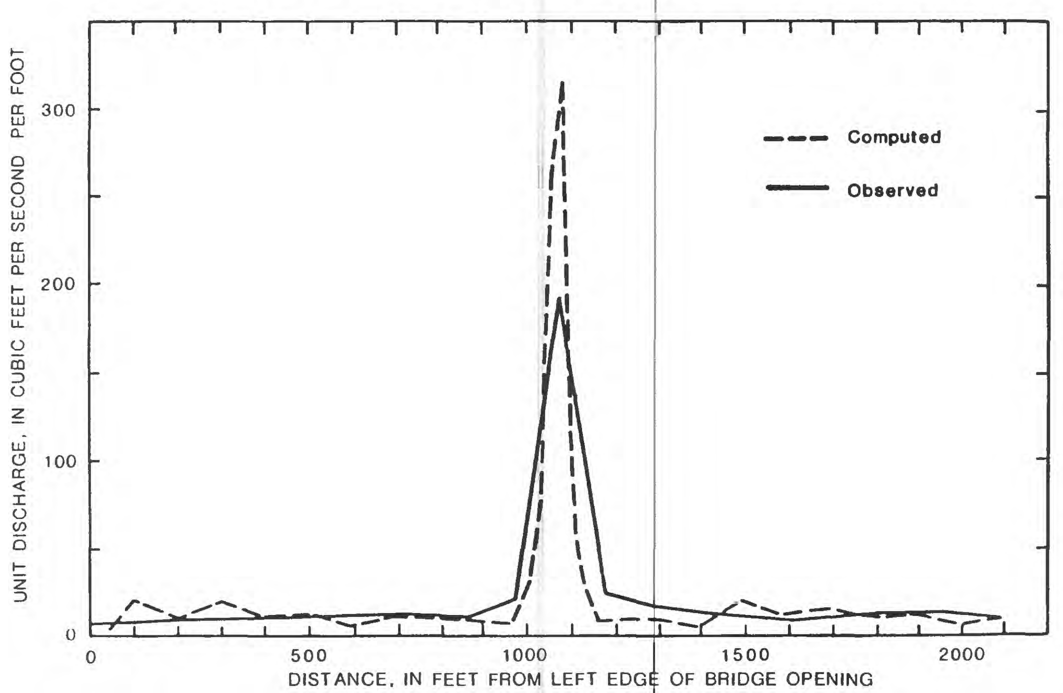

Figure 21.--Computed versus observed unit discharge of the West Pearl River at Interstate Highway 10, 1983 flood.

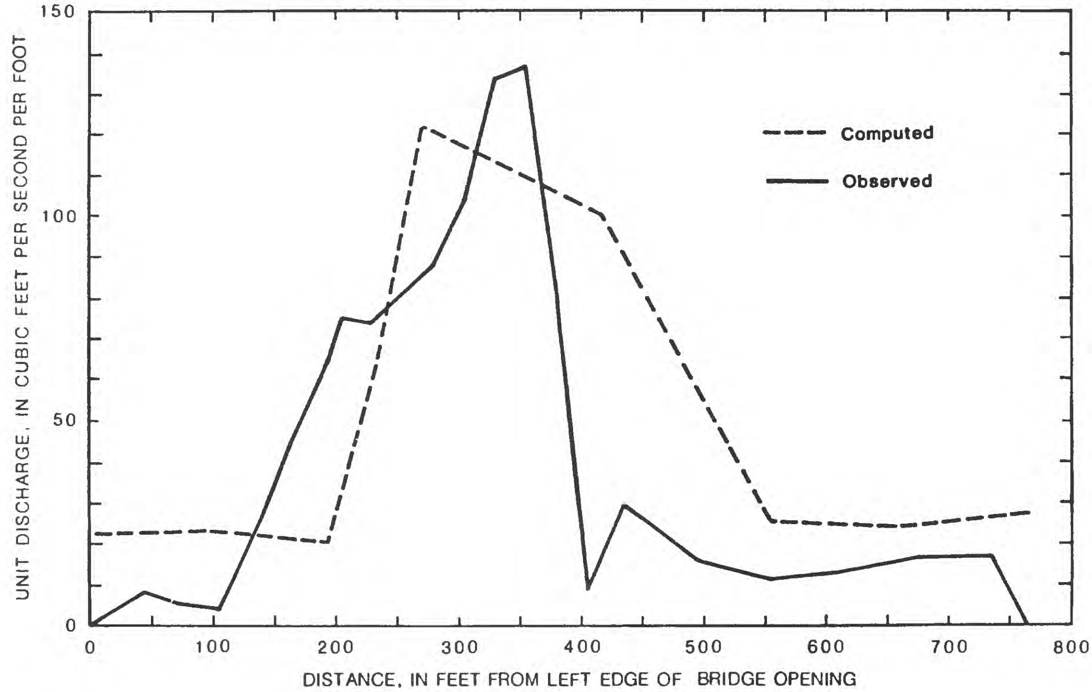

Figure 22.--Computed versus observed unit discharge of the Middle Pearl River at Interstate Highway 10, 1983 flood.

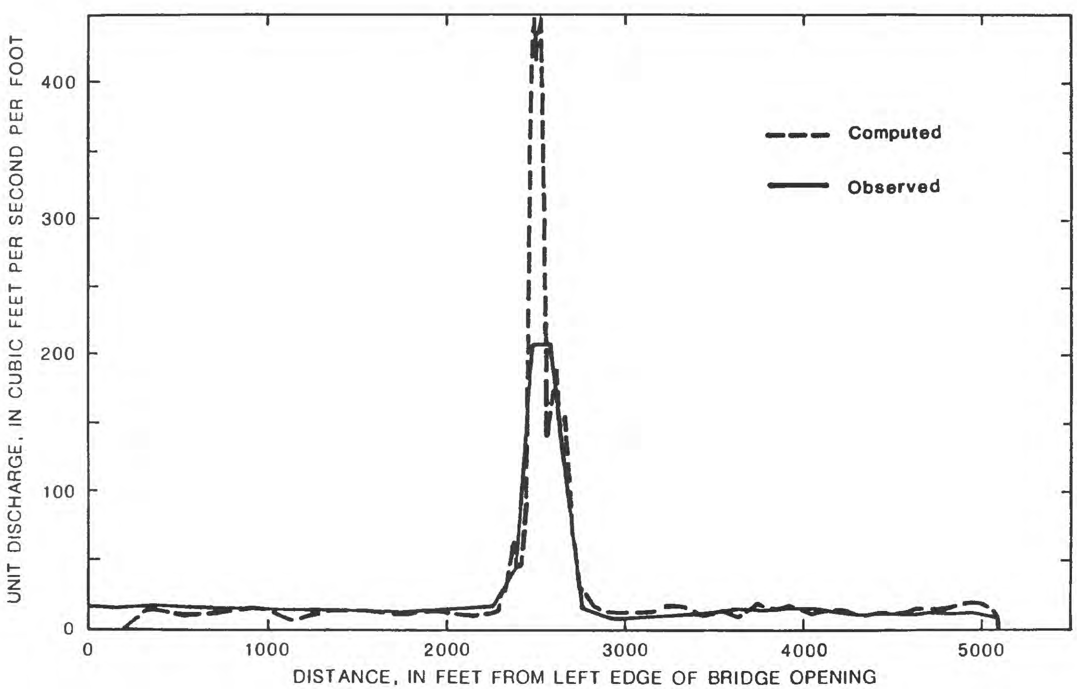

Figure 23.--Computed versus observed unit discharge of the Pearl River at Interstate Highway 10, 1983 flood.

computed and observed values for the 1983 event, a significant degree of confidence can be placed in the ability of the model to simulate other flow conditions reasonably close to those of the calibration and verification events.

Application

Following calibration and verification (highway embankments in place), the model was applied to simulate both the 1980 and 1983 floods without the U.S. 90 embankments in place. All sections of the U.S. 90 embankments between the West Pearl River bridge and the Pearl River bridge were removed from the flood plain in the model by adding elements to the network in the areas occupied by the embankments. Overbank areas within the bridge openings were assigned discharge coefficients and elevations equal to the surrounding flood plain. The networks for the "with" embankment conditions and "without" embankment conditions were otherwise the same for a particular flood simulation. The upstream boundary condition of discharge and the downstream boundary conditions of water-surface elevation were the same for the "with" and "without" simulations for a particular flood. Water-surface elevation contouring for the 1980 and 1983 floods without U.S. 90 embankments in place are shown in figures 24 and 25, respectively.

ANALYSIS OF BACKWATER AND DRAWDOWN FROM U.S. HIGHWAY 90 EMBANKMENTS

The backwater or drawdown at each node in the finite-element network was determined by subtracting the water-surface elevation computed in the simulation without the U.S. 90 embankments from values computed at corresponding nodes in the simulation with the highway embankments in place. If, at a given location, the water-surface elevation with the embankments is higher than the water-surface elevation without the embankments, giving a positive difference, the effect of the embankment is backwater at that location. If the water-surface elevation with the embankments is lower than the water-surface elevation without the embankments, giving a negative difference, the effect of the embankments is drawdown. In general, backwater occurs upstream of highway embankments and drawdown occurs downstream of embankments; however, in wide flood plains with multiple openings, complex variations of this general effect can occur.

April 1980 Flood

Computed water-surface profiles for the east and west edges of the flood plain for the embankment existing and removed are shown in figure 26. Computed lines of equal backwater and drawdown at the U.S. 90 crossing are shown in figure 27. The maximum backwater in the study area for the April 1980 flood was 1.0 ft on the upstream side of the embankment at the intersection of U.S. 90 and U.S. 190. This location is also the intersection of the right edge of the flood plain and the U.S. 90 embankment; therefore, maximum backwater on the west edge of the flood plain was also 1.0 ft. Maximum backwater on the east edge of the flood plain was 0.3 ft near Pearlington, Miss., about

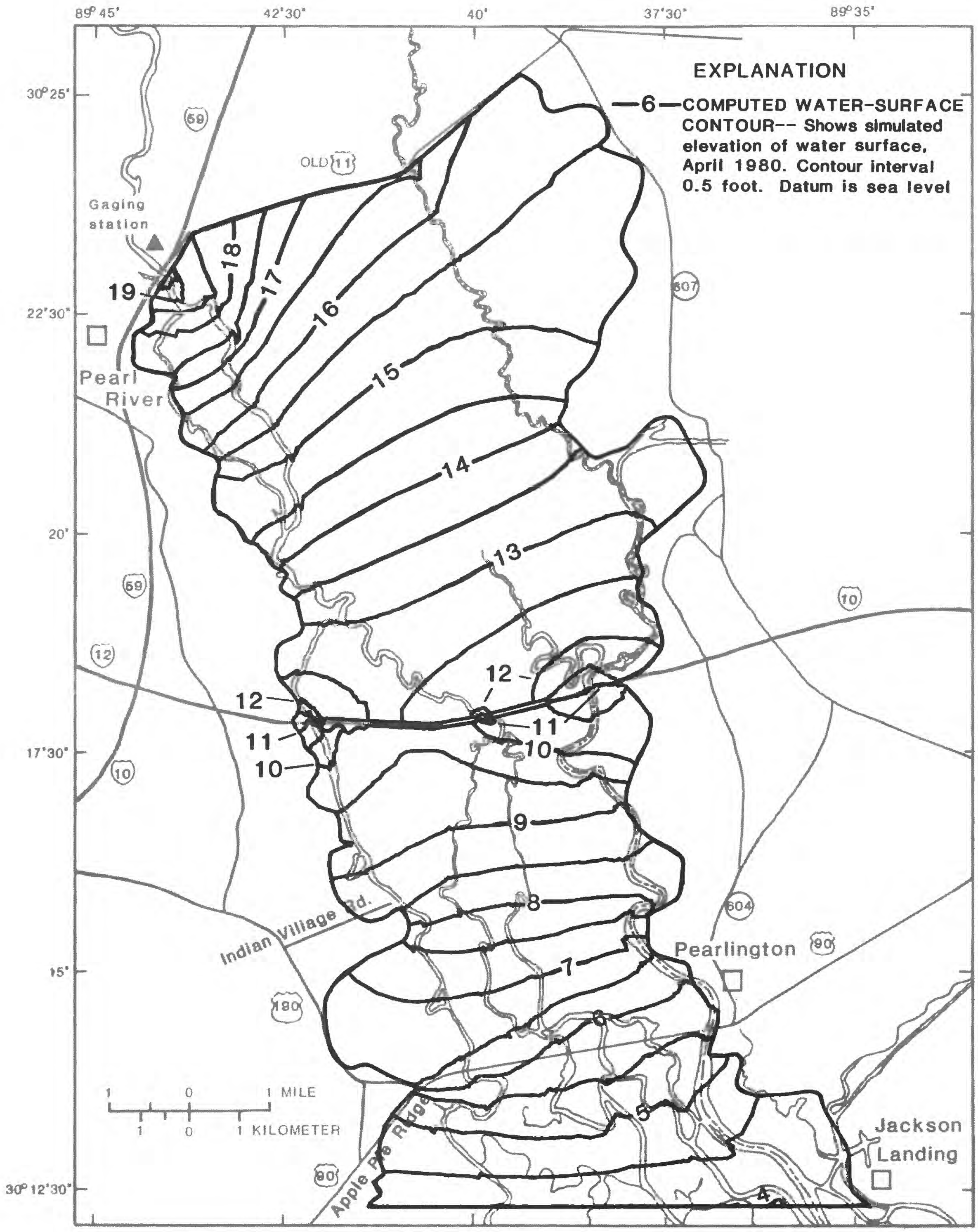

Figure 24.--Water-surface elevation contours, 1980 flood, with the U.S. Highway 90 embankment removed.

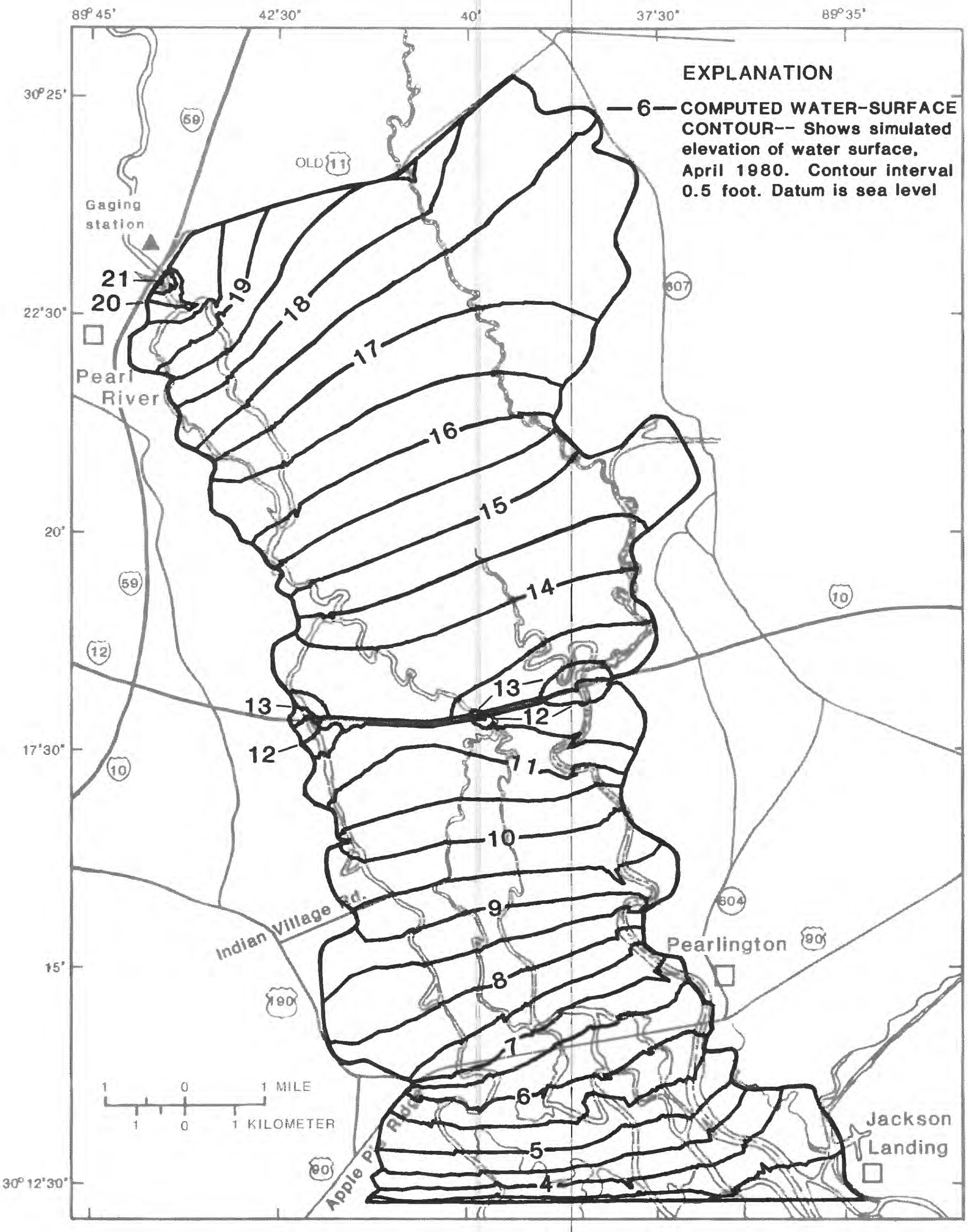

Figure 25.--Water-surface elevation contours, 1983 flood, with the U.S. Highway 90 embankment removed.

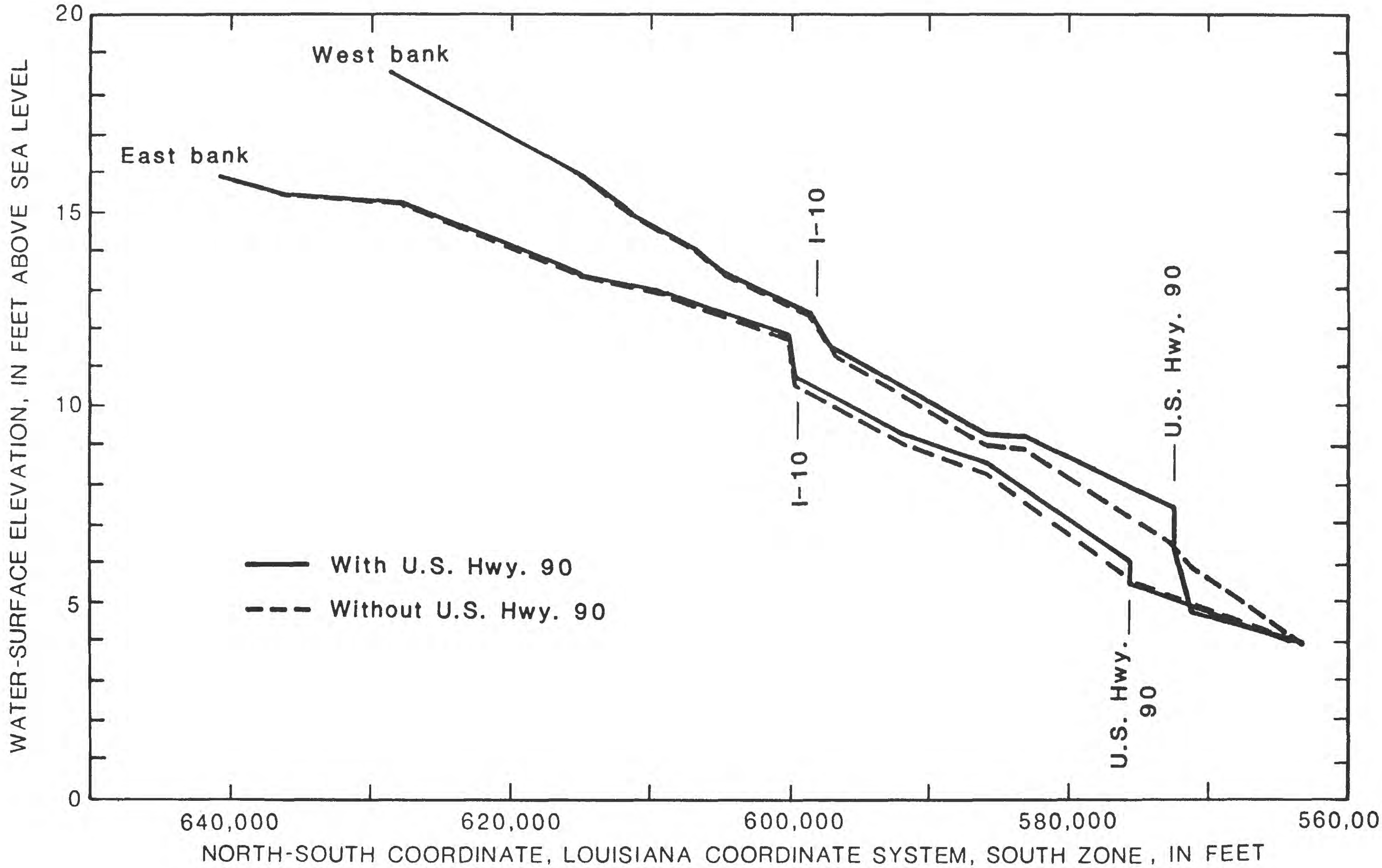

Figure 26.--Water-surface profiles with and without U.S. Highway 90, east and west banks, 1980 flood.

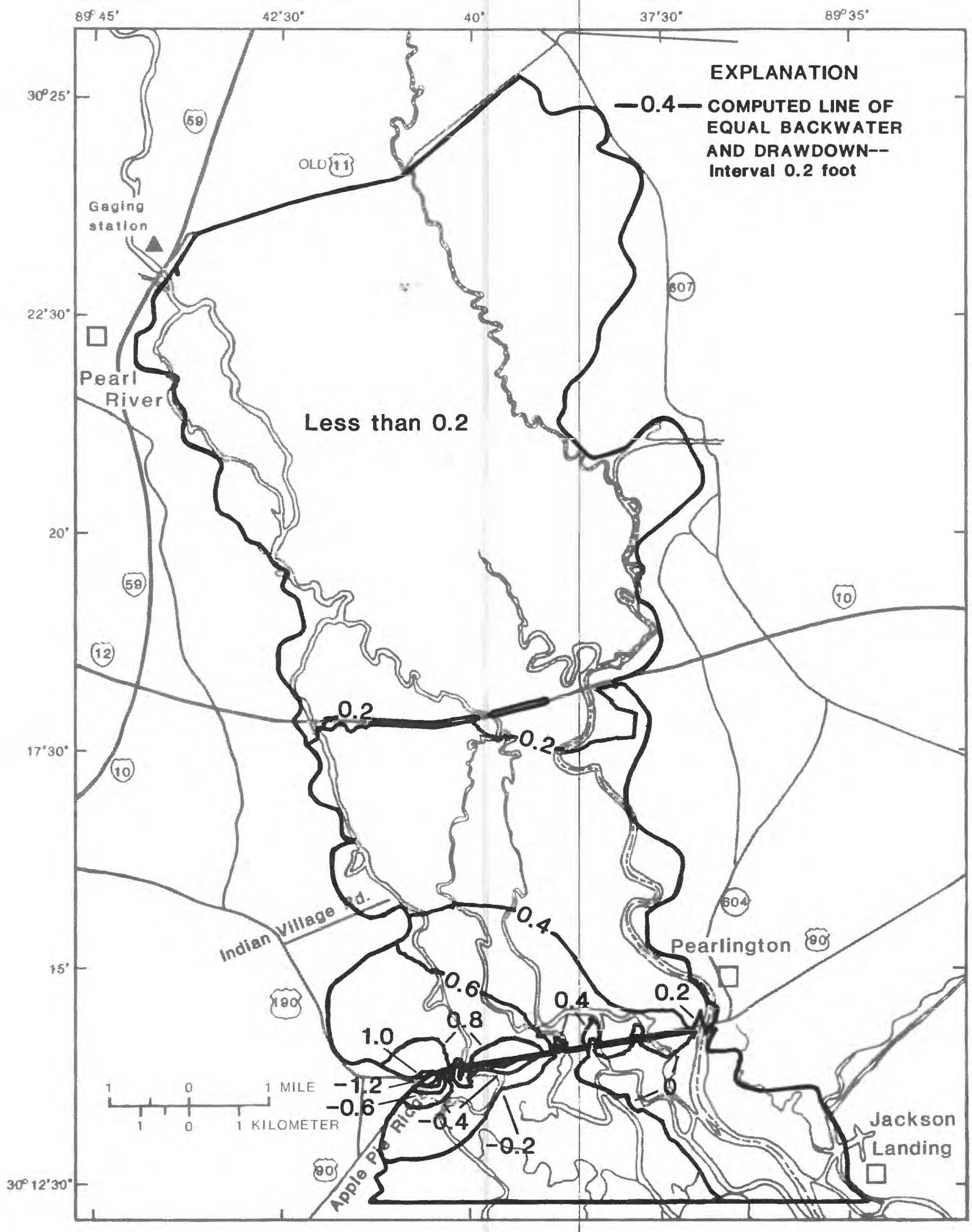

Figure 27.--Backwater caused by U.S. Highway 90, 1980 flood.

7,000 ft upstream of the highway crossing. Backwater is greater on the west bank than on the east, but decreases at a faster rate, moving upstream, on the west bank. Backwater extends upstream through I-10, with values of 0.2 ft near I-10. An area of drawdown is located downstream of the U.S. 90 embankment west of the West Middle Pearl River. Backwater of 0.1 ft extends slightly downstream of the U.S. 90 embankment at the East Middle and Pearl Rivers. The computed discharge at the highway crossing with and without the embankments in place is shown in table 9.

Table 9.--Computed discharge at the U.S. Highway 90 crossing with and without the embankments in place, 1980 flood

Subsection (from east side to west side of flood plain)	Discharge with highway embankments (cubic feet per second)	Discharge without highway embankments (cubic feet per second)
Pearl River bridge opening---------	72,700	55,300
Overbank--------------------------	0	13,800
East Middle River bridge opening---	19,900	11,000
Overbank--------------------------	0	8,300
Middle River bridge opening--------	27,100	15,900
Overbank--------------------------	0	5,600
West Middle River bridge opening---	36,300	19,400
Overbank--------------------------	0	29,200
West Pearl River bridge opening----	22,900	9,900
Overbank--------------------------	0	14,900
Totals[1]-------------------------	179,000	183,000

[1] Totals are rounded to the nearest thousand.

April 1983 Flood

Computed water-surface profiles for the east and west edges of the flood plain for the embankment existing and removed are shown in figure 28. Computed lines of equal backwater and drawdown at the U.S. 90 crossing are shown in figure 29. The maximum backwater in the study area was 1.2 ft on the upstream side of the U.S. 90 embankment midway between the West Pearl and West Middle Pearl Rivers. Maximum backwater on the west edge of the flood plain was 1.2 ft near the intersection of U.S. 90 and U.S. 190. Maximum backwater on the east edge of the flood plain was 0.3 ft, located 5,000 ft upstream of the U.S. 90 embankment. As in the 1980 flood, backwater is greater on the west side of the flood plain than on the east side, but decreases at a greater rate in the upstream direction until the uniform distribution of back-

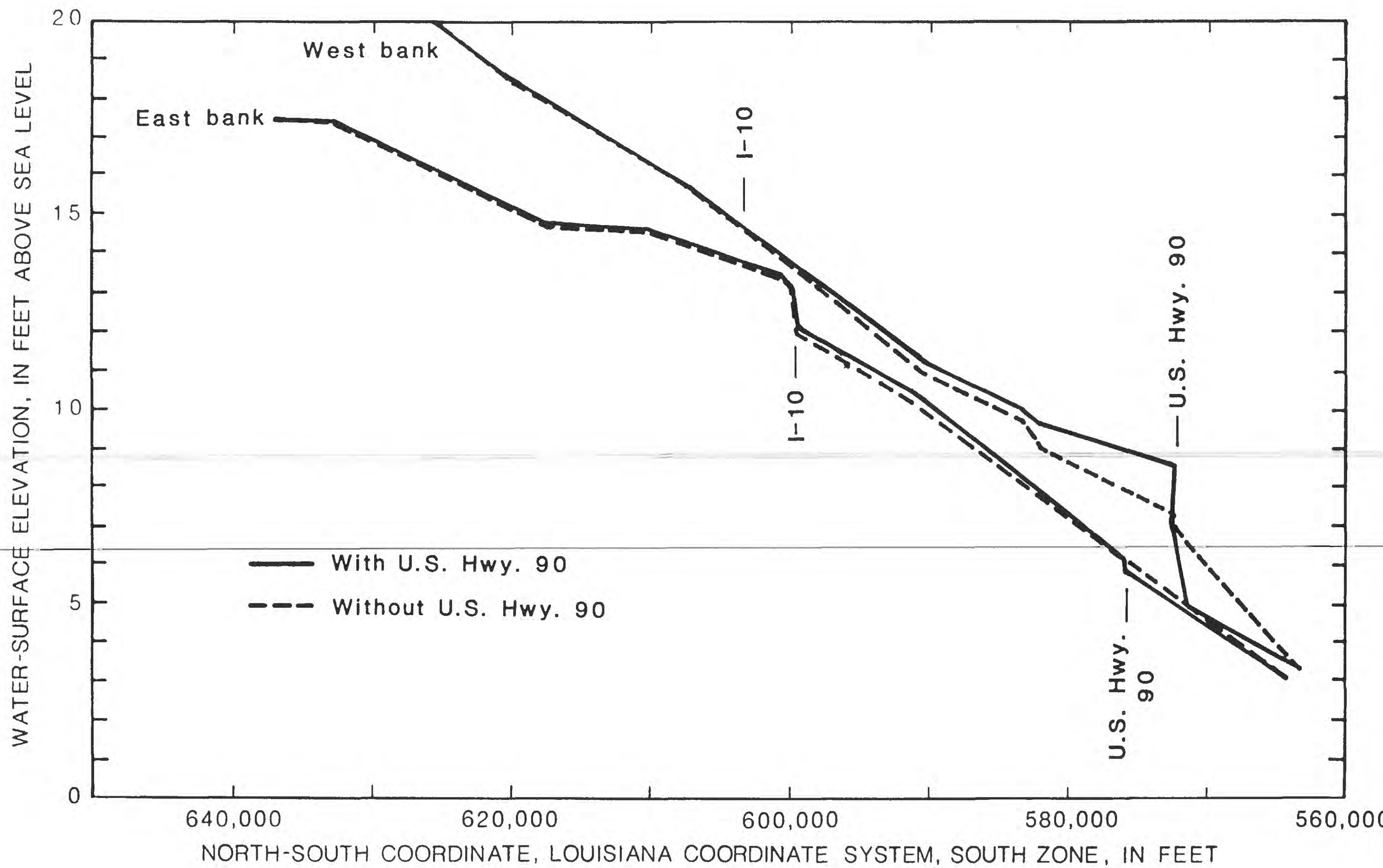

Figure 28.--Water-surface profiles with and without U.S. Highway 90, east and west banks, 1983 flood.

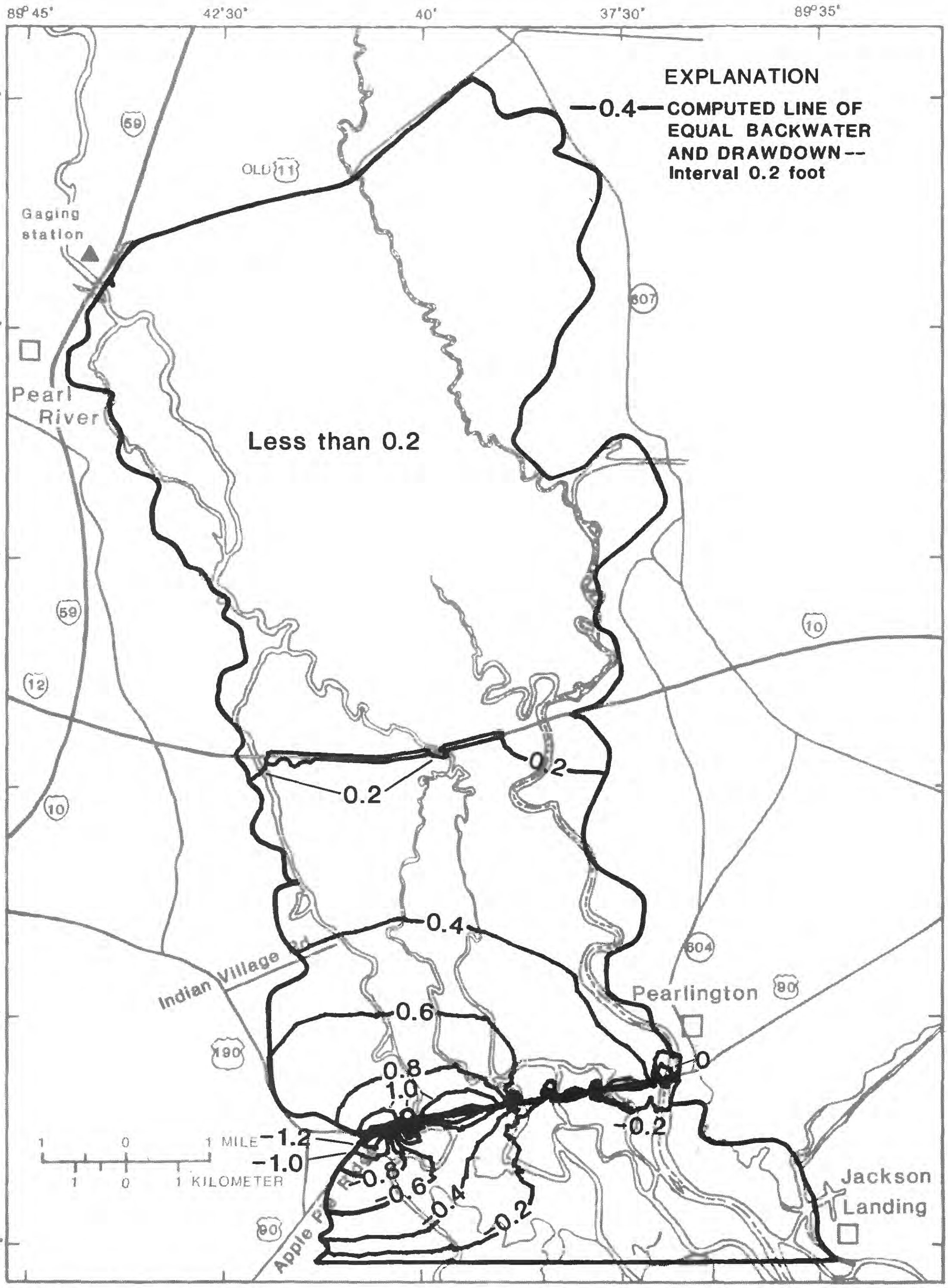

Figure 29.--Backwater caused by U.S. Highway 90, 1983 flood.

water of 0.2 ft occurs across the flood plain near I-10. A large area of drawdown is located downstream of U.S. 90 on the west side of the flood plain. A smaller area of drawdown is located downstream of U.S. 90 near the Pearl River bridge opening. The maximum drawdown below U.S. 90 is 1.2 ft, located downstream of the intersection of U.S. 90 and U.S. 190. The computed discharge at the highway crossing with and without the embankments in place is shown in table 10.

Table 10.--Computed discharge at the U.S. Highway 90 crossing with and without the embankments in place, 1983 flood

Subsection (from east side to west side of flood plain)	Discharge with highway embankments (cubic feet per second)	Discharge without highway embankments (cubic feet per second)
Pearl River bridge opening---------	81,600	64,800
Overbank---------------------------	0	18,900
East Middle River opening----------	25,900	14,000
Overbank---------------------------	0	11,700
Middle River bridge opening--------	35,700	20,000
Overbank---------------------------	0	7,800
West Middle River bridge opening---	46,900	23,700
Overbank---------------------------	0	43,200
West Pearl River bridge opening----	29,900	14,000
Overbank---------------------------	0	23,800
Totals-------------------------	220,000	[a]242,000

[a] Total is rounded to the nearest thousand.

Discussion of Backwater Effects

The backwater, due to U.S. 90 embankments, during the 1983 flood was generally slightly greater than the backwater during the 1980 flood. Near the intersection of U.S. 90 and U.S. 190, backwater is approximately 0.2 ft greater. On the east side of the flood plain and near I-10 backwater is approximately equal to the 1980 simulations, even though the 1983 flood had a larger discharge and higher water-surface elevation. This is attributed to the complexity of the flow in the study reach at high stages which is caused primarily by the presence of the two multiple-opening highway crossings. Not only do these structures cause an increase (or decrease) in water-surface elevation, but they also affect the flow distribution within the flood plain and river channels. The effects of redistribution (location of openings) may be larger than the effects of any reductions in flow areas (areas of the openings). A combination of natural and man-made factors caused most of the flow to enter the study reach on the west side of the flood plain and to leave on the east side. This transfer of flow is a natural response, given the flood plain geometry without highway crossings. Manmade highway crossings tend to expedite this transfer of flow. (See Lee and others, 1983, and tables 6, 7, and 9 in this report.)

One result of this redistribution of flow is the area of drawdown just downstream of U.S. 90 near the West Pearl River bridge opening. (See figs. 27 and 29.) Because of the faster flow transfer from the west side to the east side of the flood plain, with highway embankments in place, U.S. 90 increases water-surface elevations on the western upstream side of the roadway in this area while causing a relatively large decrease in water-surface elevations on the western downstream side. The magnitude of backwater and drawdown at the embankment on the east side of the flood plain is considerably less than for the west side of the flood plain.

The flow distribution at the U.S. 90 crossing is similar for both floods. The effect of the U.S. 90 crossing on the redistribution of flow, however, seems to be less for the 1983 flood than the 1980 flood. This may be due in part to the roadway overflow at I-10 during the 1983 event and also in part to the naturally more rapid transfer of flow at higher stages. A detailed analysis of the effects of U.S. 90 and I-10 on the distribution of flow within the study reach for a different peak flow is beyond the scope of this report.

The fact that flow over U.S. 190 and flow in the area of Indian Village Road was not considered in modeling the 1980 flood may have slightly affected the computed water-surface elevations both with and without the U.S. 90 embankments in place. Although flow in these two places was quite small in comparison to the total flow in the flood plain, the inclusion of this flow in the model would tend to increase water-surface elevations somewhat in the area along the west side of the flood plain downstream of Indian Village Road. The backwater computed for the 1980 flood in this area just downstream of Indian Village Road may be slightly underestimated, perhaps by as much as 0.1 ft.

SUMMARY AND CONCLUSIONS

A two-dimensional finite-element surface-water flow model was used to determine the effect of U.S. 90 embankments on water-surface elevations in the Pearl River flood plain during the floods of April 1980 (a 50-year event) and April 1983 (a 200-year event). The model was calibrated using data collected for the 1980 flood. The 1983 flood was then simulated and computed results were compared to observed values. Because of the close agreement between computed and observed water-surface elevations and discharges for the 1983 event, the validity of the model was confirmed. The model was then applied to determine the backwater caused by U.S. 90 during both the 1980 and 1983 floods by simulating these events with the highway embankments removed from the model.

The maximum backwater produced by U.S. 90 during both the 1980 and 1983 floods was slightly more than 1.0 ft and was located along the upstream side of the roadway embankment. For both events, the distribution of backwater is not uniform. Backwater is at least three times greater on the west bank than on the east bank near U.S. 90. Backwater decreases at a greater rate, moving upstream, along the west side of the flood plain than along the east side and is approximately the same on both sides at I-10. The backwater at I-10 was 0.2 ft, approximately, for both the 1980 and 1983 floods. At U.S. 90, the maximum backwater for the 1980 flood was approximately 1.0 ft on the west side of the flood plain near the intersection of U.S. 90 and U.S. 190. The maximum backwater on the east side of the flood plain was 0.3 ft, near Pearlington, Miss.

For the 1983 flood, the maximum backwater at U.S. 90 was 1.2 ft, located on the upstream side of the embankment between the West Pearl and West Middle Rivers. Maximum backwater on the west edge of the flood plain was also 1.2 ft. The maximum backwater of 0.2 ft occurs near Pearlington, Miss., on the east side of the flood plain.

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