SIMULATION OF FLOW AND TRANSPORT IN THE LOWER MISSISSIPPI RIVER, LOUISIANA

By Philip B. Curwick

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CONVERSION FACTORS AND ABBREVIATIONS

For the convenience of readers who prefer to use metric (International System) units rather than the inch-pound units used in this report, values may be converted by using the following factors:

Multiply inch-pound unit	By	To obtain metric unit
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
foot (ft)	0.3048	meter (m)
foot per second (ft/s)	0.3048	meter per second (m/s)
square foot per second (ft^2/s)	0.0929	square meter per second (m^2/s)
mile (mi)	1.609	kilometer (km)

Temperature in degrees Celsius (°C) can be converted to degrees Fahrenheit (°F) as follows: °F = 1.8 X °C + 32.

National Geodetic Vertical Datum of 1929 (NGVD of 1929): A geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada, formerly called "Mean Sea Level."

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ABSTRACT

The fully dynamic, one-dimensional equations of unsteady open-channel flow and convective diffusion have been solved uncoupled in numerical models of the lower Mississippi River from Tarbert Landing, Mississippi, to Venice, Louisiana. The reach extends 295 miles. The flow model uses a weighted, four-point, implicit, finite-difference approximation for solution of the unsteady flow equations. The transport model uses an explicit finite-difference approximation of the continuity of mass equation for solution in a Lagrangian coordinate system.

The flow model was calibrated with 3 months of stage and discharge data from the flood of 1979. The root-mean-square errors for stage ranged from 0.26 to 1.13 feet. Average root-mean-square error for stage was 0.47 foot on the basis of results from seven gaging stations located in the study reach. The root-mean-square errors for discharge ranged from 54,230 to 176,200 ft^3/s (cubic feet per second). Average root-mean-square error for the limited discharge data was 113,400 ft^3/s for the 1979 flood. The flow model was verified with 3 months of stage and discharge data for the flood of 1983. The root-mean-square errors for stage ranged from 0.22 to 0.63 foot during this flood. Average root-mean-square error for stage was 0.40 foot. The root-mean-square errors for discharge ranged from 49,950 to 106,000 ft^3/s. The average root-mean-square error for discharge was 86,280 ft^3/s for the flood of 1983.

The transport model was calibrated with dye-tracer data from a dye injection of 1965 and verified with dye-tracer data from a dye injection of 1974. Calibration resulted in a range of root-mean-square errors of dye concentration from 0.14 to 0.22 ppb (parts per billion) and averaged 0.18 ppb based on three time-concentration curves. The verification resulted in a range of root-mean-square errors of dye concentration from 0.067 to 0.13 ppb on the basis of five time-concentration curves and averaged 0.087 ppb.

Numerical experiments were used to perform a sensitivity analysis of the controlling parameters of both models. Computed stages and discharges are insensitive to the length of the time step and average wind conditions. The computed stages are insensitive to changes in flow-resistance coefficients, but discharges are highly sensitive to these changes. A rigid boundary assumption for channel cross sections is reasonable. Computed time-concentration curves are sensitive to changes in the dispersion factor.

These flow and transport models provide a convenient and economical framework to analyze the hydrology and water quality of the lower Mississippi River.

INTRODUCTION

The lower Mississippi River has been, and continues to be, the object of intensive study (Everett, 1971; Robbins, 1976; Wells, 1980; Grayman, 1985; and Demas and Curwick, 1986). Water and sediment carried by the river are viewed as both a favorable and unfavorable natural resource. Economically optimum and environmentally sound management of river water and sediment requires more quantitative information of suspended-sediment and associated chemical transport characteristics of the system. Such knowledge is important for management of diverted Mississippi River water and sediment for coastal land accretion strategies being considered. Mathematical modeling of flow and transport processes in rivers is rapidly becoming the accepted engineering tool necessary to provide such information and to furnish feedback for management alternatives.

Purpose and Scope

In late 1982, the U.S. Geological Survey, in cooperation with the Louisiana Department of Transportation and Development, began a detailed study of suspended-sediment and associated chemical transport in the lower Mississippi River from Tarbert Landing, Miss., to Venice, La. (pl. 1). An integral component of that study is the development of a suspended-sediment and associated chemical transport model. The hydraulic data and transport framework necessary for these computations were derived from two numerical models. The purpose of this report is to describe the streamflow and transport models used for those computations. Data collected in previous studies were updated for development of the model.

Acknowledgments

Appreciation is expressed to the U.S. Army Corps of Engineers, New Orleans District, for providing stage and cross-sectional data for use in this study.

DESCRIPTION OF LOWER MISSISSIPPI RIVER STUDY REACH

The study reach in this investigation is a 295-mile stretch of the main stem of the lower Mississippi River between Tarbert Landing, Miss., and Venice, La. A map of the study reach is presented on plate 1. Locations on the lower Mississippi River main stem are referenced by a river-mile system, measured in terms of river miles above Head of Passes, La., a trifurcation point on the main stem in the Mississippi River delta (pl. 1).

As the Mississippi River wends its way to the Gulf of Mexico, it meanders between deep bends and shallower crossings. Channel geometry (fig. 1) is typical of a meandering confined channel that occurs on lower, gentler slopes towards the river mouth. Channel cross-sectional area generally increases with downstream distance. The local mean depth of flow varies from a minimum of approximately 20 ft at some crossings to a maximum bend depth of about 200 ft. The thalweg, line of maximum depth, is below the National Geodetic Vertical Datum of 1929 (NGVD of 1929) throughout the study reach and remains below the NGVD of 1929 as far upstream as mile 350. The average channel-bottom slope is a mild $3.4 X 10^{-2}$ ft/mi.

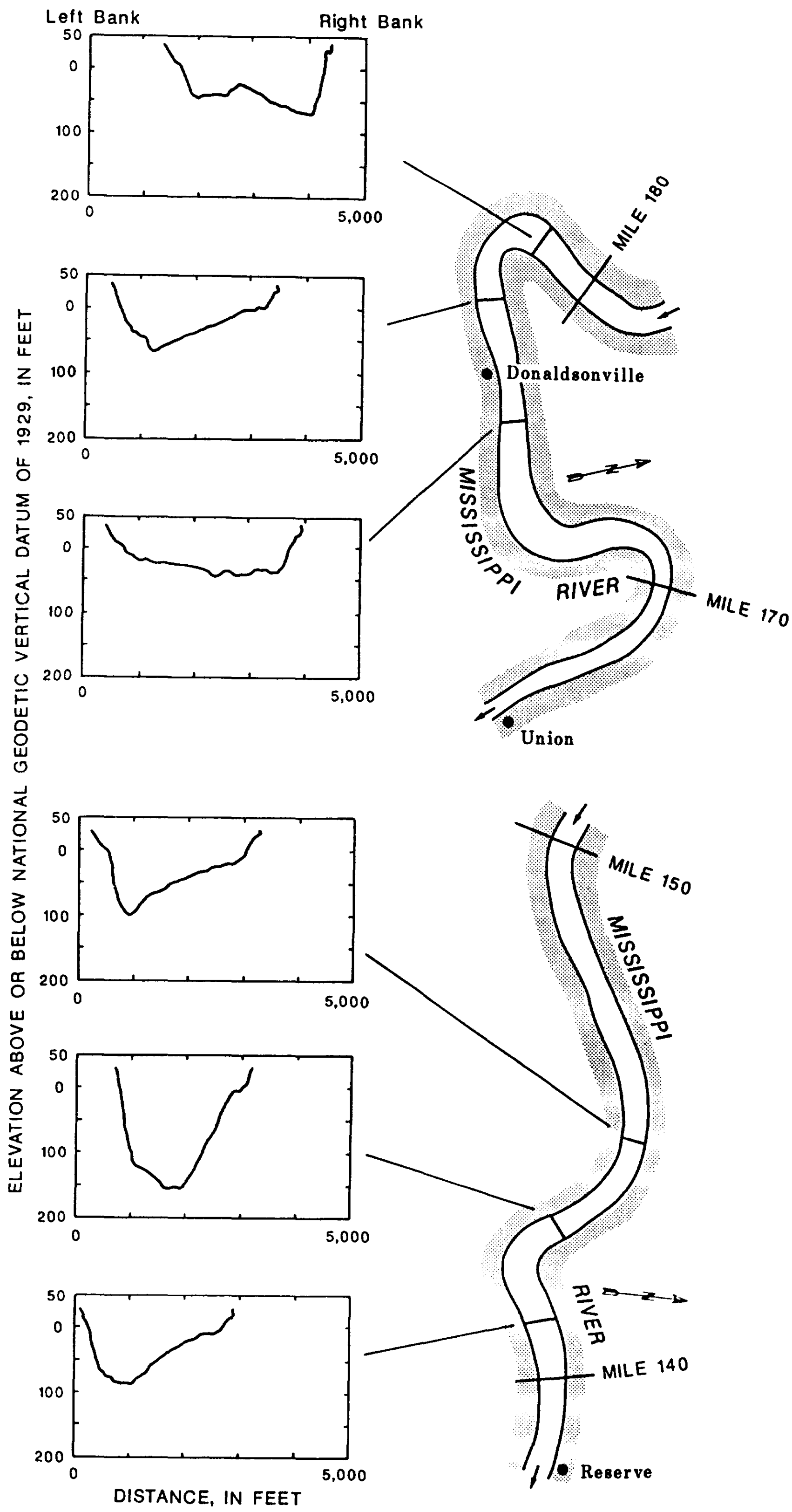


Figure 1.—Representative cross sections of the lower Mississippi River.

Streamflow in the lower Mississippi River is affected by diversions to the Atchafalaya River through the Old River control structure (pl. 1). The Atchafalaya River is a historic route for spilling floodwaters of the Mississippi (by way of Old River) and Red Rivers into the Gulf of Mexico. A study conducted in 1953, by the U.S. Army Corps of Engineers, indicated that if left completely to its own devices, the lower Mississippi River would have changed its course toward the Gulf of Mexico to a route by way of the Old and Atchafalaya Rivers sometime between 1965 and 1975 (Lower Mississippi Region Comprehensive Study Coordinating Committee, 1974). The Old River control structure was constructed to reduce that possibility and now restricts the flow entering the Atchafalaya River to approximately 30 percent of the total flow of the lower Mississippi River main stem.

In addition to the Old River control structure diversion, streamflow is also diverted intermittently at two floodways in the study reach of lower Mississippi River. During extreme floods, water can be diverted through the Morganza Floodway and the Bonnet Carre′ Floodway (pl. 1). Other than these man-made structures, there are no tributaries or distributaries significantly affecting streamflow until the river reaches Head of Passes, La.

The flow pattern in the lower Mississippi River is usually unidirectional, turbulent, and pulsating. Exceptions occur during long periods of extreme low flow and during hurricane surges, when a significant length of the lower study reach is affected by bidirectional flow. Because the thalweg of the lower Mississippi River is below the NGVD of 1929, saltwater intrudes some distance upstream during periods of low flow. The extent of saltwater intrusion depends primarily on river discharge; however, flow duration, wind velocity and direction, tides, and river-bed conditions also influence the intrusion of saltwater upstream (Wells, 1980, p. 40-44). For flood flows above 200,000 ft^3/s, saltwater intrusion was assumed to be insignificant and was not included in the modeling.

Turbulent flow is always characterized by local eddying, which results in pulsations in the velocity at a fixed point. Figure 2 shows the pulsations observed at a fixed point in the lower Mississippi River at Baton Rouge, La., over a period of 6 hours. The discharge was approximately 400,000 ft^3/s. Velocities are shown to vary plus or minus 20 percent about the mean in as little as 15 minutes. Despite these temporal variations, flow is near uniformly distributed laterally at most locations and for all flow regimes. An example of the lateral-flow distribution is shown in figure 3.

HISTORIC DATA COLLECTION

Stage, discharge, and other site-specific project-related data on the lower Mississippi River are cooperatively collected and published by the U.S. Geological Survey, U.S. Army Corps of Engineers, and National Weather Service. The locations of the gaging stations are shown on plate 1 and listed in table 1. Stage data are the most predominant type of data collected and published.

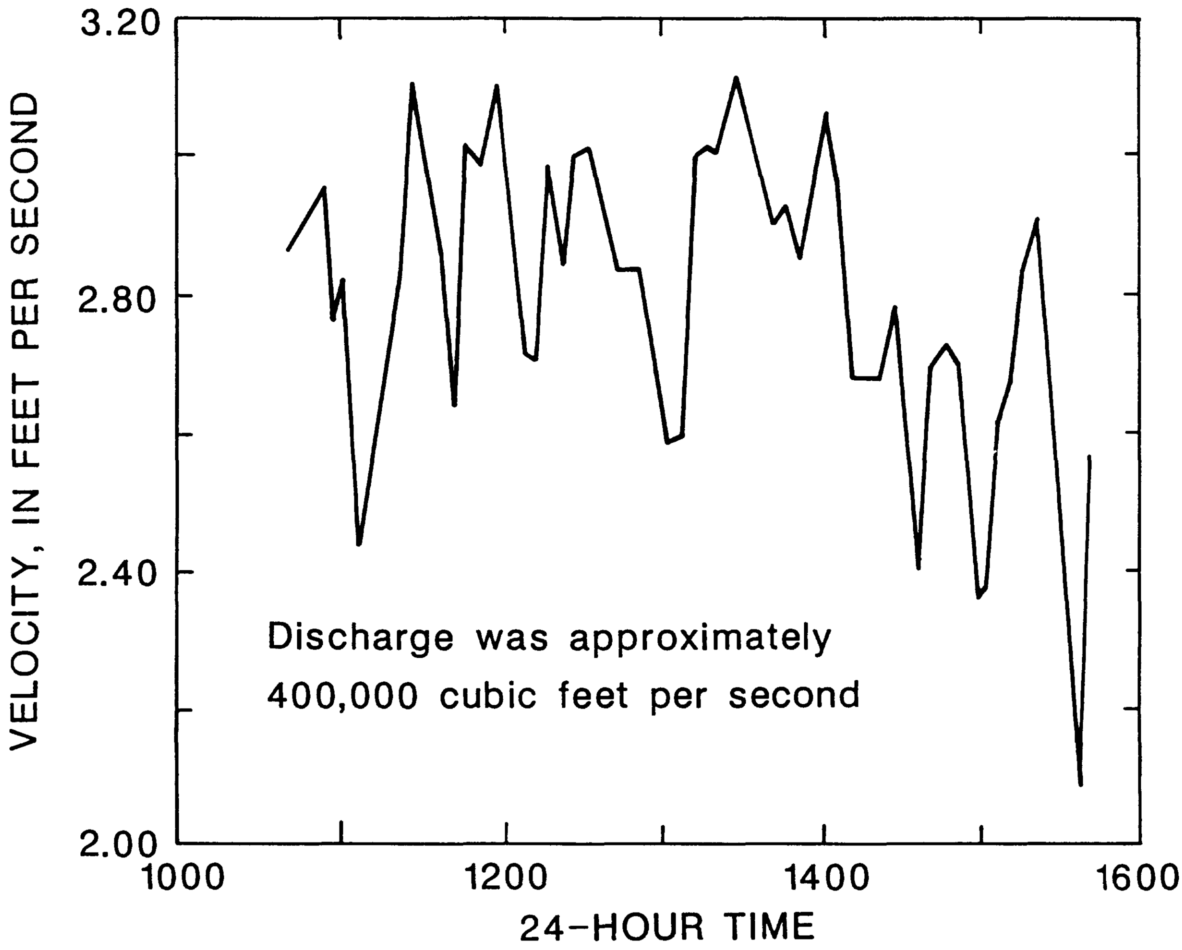


Figure 2.—Velocity fluctuations measured on the Mississippi River at Baton Rouge, Louisiana, on September 17, 1974.

River stage changes only slightly from day to day but can change as much as eightfold during a major flood. A major flood can last from 2 to 5 months. Discharge behaves similar to stage in terms of day to day fluctuations, but can change ninefold during a major flood. No single-valued rating between stage and discharge exists for the river. The rating is complicated by combined effects of unsteady flow, sediment load, changing bed forms, and backwater (tide in lower reaches).

Daily discharge records are published for only one site (Tarbert Landing) in the study reach. Instantaneous discharge is measured by the Corps about every 3 days at Tarbert Landing, Miss. Stage measured at Red River Landing, La., mile 302.4, (just downstream from Tarbert Landing) is used to develop the short-term relation between stage and discharge. This relation is used to construct the daily discharge records between measurements for Tarbert Landing. The records are published annually by the Survey. The Survey also publishes instantaneous discharge data at several sites in conjunction with flood measurements and water-quality studies.

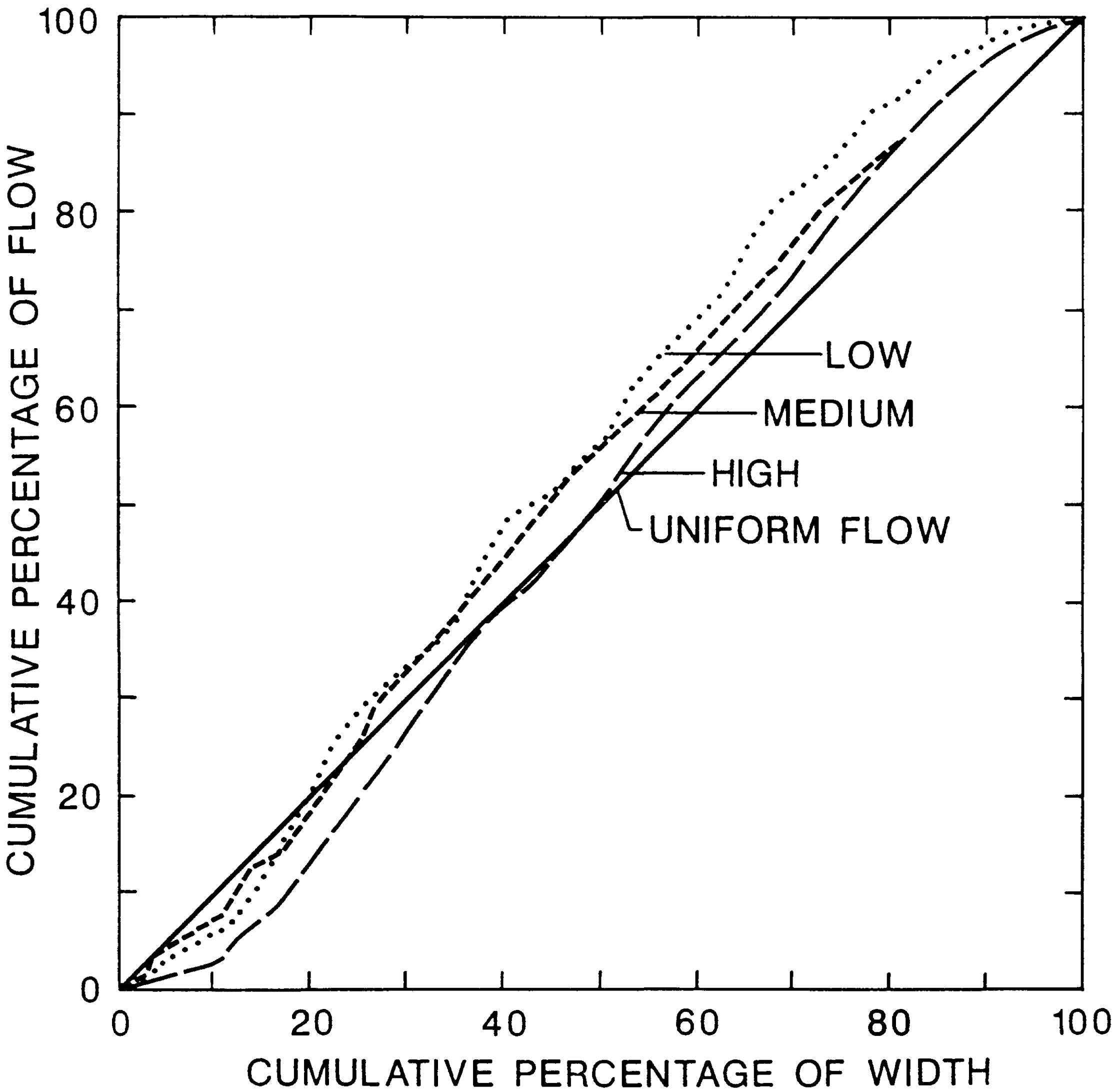


Figure 3.--Lateral flow distribution of the Mississippi River at Belle Chasse, Louisiana, for low, medium, and high discharges.

In addition to the streamflow information that is available, the Survey has performed several studies aimed at identifying the time-of-travel and dispersion characteristics of solutes in the lower Mississippi River. Based on dye-tracer studies performed in 1965 (Stewart, 1967), 1969 (Everett, 1971), 1974 (Martens and others, 1974), 1975 (Calandro, 1976), and 1976 (Calandro, 1977), the Survey has estimated time-of-travel and dispersion characteristics under varying flow conditions from the Arkansas-Louisiana State line to West Pointe a la Hache, La. Estimated travel times for the leading edge, peak, and trailing edge of a tracer cloud are summarized graphically by Wells (1980). An empirical technique for predicting the peak concentration of a conservative contaminant at downstream sites based on discharge, travel time, and quantity of pollutant is also presented by Wells (1980).

Table 1.--*List of selected gaging stations and data-collection sites on the lower Mississippi River*

[Eight-digit numbers are U.S. Geological Survey station numbers; seven-digit numbers are U.S. Army Corps of Engineers station numbers]

Station number	Station name	River mile	Data type		
			Stage	Discharge	Dye
07295100	Mississippi River at Tarbert Landing, Miss.	306.3		X	
07373290	Mississippi River at Red River Landing, La.	302.4	X		
07373310	Mississippi River at Bayou Sara, La.......	265.4	X		
07374000	Mississippi River at Highway 190, Baton Rouge, La............	234.4		X	X
07374050	Mississippi River at Port Allen Lock, La..	228.4	X		
07374120	Mississippi River at Ferry, at Plaquemine, La...................	208.0			X
07374200	Mississippi River at Donaldsonville, La...	175.4	X		
07374220	Mississippi River at Union, La............	167.5			X
07374270	Mississippi River at College Point, La....	157.4	X		
07374320	Mississippi River at Reserve, La..........	138.7	X		
07374510	Mississippi River at Carrollton Avenue at New Orleans, La......	102.8	X		X
07374520	Mississippi River at Chalmette, La........	91.0	X		
0138604	Mississippi River near Braithwaite, La.	76.6	X		
07374525	Mississippi River at Belle Chasse, La.....	76.0		X	X
0139004	Mississippi River at Alliance, La.........	62.5	X		
07374530	Mississippi River at West Pointe a la Hache, La............	48.7	X		X
0142004	Mississippi River at Port Sulphur, La.....	39.3	X		
07374535	Mississippi River at Empire, La...........	29.5	X		
07374550	Mississippi River at Venice, La...........	10.7	X		

FORMULATION OF FLOW AND TRANSPORT MODELS

The fundamental notions and hypotheses used in the mathematical models of flow and transport in the lower Mississippi River, are formalized in the one-dimensional equations of unsteady open-channel flow and convective-diffusion transport.

Unsteady Flow Equations

The derivation of the equations of unsteady flow has been reported previously (Strelkoff, 1969). The equations presented by Schaffranek and others (1981) were used in this study. For a channel with negligible lateral inflow (fig. 4), these equations are the continuity equation

$$B \frac{\partial Z}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (1)$$

and the momentum equation

$$\frac{1}{gA} \left(\frac{\partial Q}{\partial t}\right) + \frac{2\beta Q}{gA^2} \left(\frac{\partial Q}{\partial x}\right) - \frac{\beta Q^2}{gA^3} \left(\frac{\partial A}{\partial x}\right)\Big|_Z + \frac{\partial Z}{\partial x} + \frac{\kappa}{A^2 R^{4/3}} Q|Q| - \frac{\tau B}{gA} U_a^2 \cos a = 0 \qquad (2)$$

The water-surface elevation, Z, and the discharge, Q, are the dependent variables. The longitudinal distance along the channel, x, and the time, t, are the independent variables. The cross-sectional area, top width, hydraulic radius, and acceleration due to gravity are given as A, B, R, and g, respectively. The wind velocity vector, U_a, makes an angle, a, with the positive x-axis. The coefficient, κ, is a function of the flow resistance coefficient, η, (similar to Manning's n), and can be expressed in the inch-pound system of units as

$$\kappa = \left(\frac{\eta}{1.49}\right)^2 \qquad (3)$$

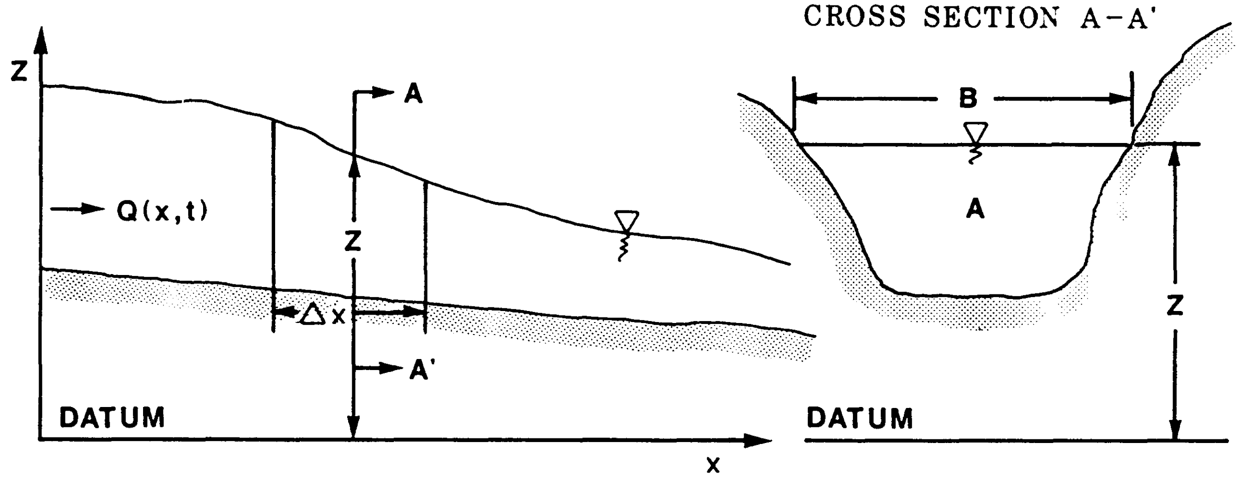
The coefficient, τ, is a dimensionless wind-resistance coefficient which can be expressed as a function of the water-surface drag coefficient, c_d, the water density, ρ, and the air density, ρ_a as

$$\tau = c_d \frac{\rho_a}{\rho} \qquad (4)$$

The coefficient, β, is known as the momentum correction coefficient and can be expressed as

$$\beta = \frac{\int u^2 dA}{U^2 A} \qquad (5)$$

where u represents the velocity of water passing through some finite elemental area, dA, U is the mean velocity in the cross section, and A is the cross sectional area as previously defined. The β coefficient is included to adjust for any nonuniform velocity distribution over the channel cross section.



EXPLANATION

Z is the water-surface elevation, in feet
x is the distance along the channel, in feet
t is the time, in seconds
B is the top width, in feet
A is the area, in square feet
Q(x,t) is the discharge as a function of x and t, in cubic feet per second
Δx is the space increment, in feet

Figure 4.--The schematic definitions of the variables of the one-dimensional equations of unsteady flow.

These equations are simplified models of extremely complex phenomena. When using this set of equations, the modeler must be aware of the physical phenomena which they do and do not incorporate. The applicability of these equations is governed by several underlying assumptions. Specifically, these assumptions are:

1. The flow is one dimensional; that is, the velocity is nearly uniform over the cross section, and the water level across the section is horizontal.

2. The streamline curvature is small, and the vertical accelerations are negligible; hence, the pressure is hydrostatic.

3. The effects of boundary friction and turbulence can be accounted for through resistance laws analogous to those used for steady-state flow.

4. The slope of the channel bottom is sufficiently mild so that the cosine of its angle with the horizontal is close to unity.

Equations 1 and 2 constitute a system of first-order, non-linear, partial-differential equations of the hyperbolic type. These equations are not amenable to analytical solution for most practical situations. The equations are solved numerically by replacing the partial-differential equations with appropriate finite difference approximations. In the flow model a weighted, four-point, finite-difference scheme is employed. The finite-difference formulation treats time derivatives of the dependent variables (water-surface elevation and discharge) as centered in space and time, and spatial derivatives of the dependent variables as centered in space and positioned in time according to a variable weighting factor. Using specified boundary and initial conditions, the system of flow equations is solved implicitly by Gauss elimination, using a maximum pivot strategy. Because the complete equations of unsteady flow are nonlinear, iterative solutions are performed within the time step to refine computed results and satisfy user-specified tolerances.

Convective-Diffusion Transport Equation

The derivation of the convective-diffusion equation has been reported previously (Fischer, 1967). The transport model documented by Jobson (1981) was used in this study. The convective-diffusion equation is solved using a coordinate system whose origin moves at the mean flow velocity (termed a Lagrangian reference frame). The convective-diffusion equation is considerably simplified by the coordinate transformation because a term containing the mean convective velocity does not appear. The continuity of mass equation for a conservative substance in a Lagrangian reference frame is

$$\frac{\partial C}{\partial t} - \frac{\partial}{\partial \xi} [D_x \frac{\partial C}{\partial \xi}] = 0 \qquad (6)$$

where C is the cross-sectional mean concentration, D_x is longitudinal dispertion coefficient, and ξ is the distance from the parcel. The Lagrangian distance coordinate is given as

$$\xi = x - x_o - \int_{t_o}^{t} U dt' \qquad (7)$$

where x is the Eulerlain distance coordinate along the river as previously defined in the flow equations, U is the cross-sectional mean flow velocity also as previously defined in the flow equations, t′ is a dummy variable of integration, and x_o is the location of the parcel at time, t_o. By setting $\xi = 0$ for a particular parcel at any time, the movement of a parcel can be tracked in the Eulerian distance coordinate system.

Integrating equation 6 gives

$$C = C_o - \int_{t_o}^{t} \frac{\partial}{\partial \xi} [D_x \frac{\partial c}{\partial \xi}] dt' \qquad (8)$$

where C_o is the concentration of the parcel at time, t_o, and C is the concentration after a time lapse of t. The formulation is completed by approximating the distance between parcels, $\partial\xi$, as the velocity times the time step size, Δt, and expressing the integral of the dispersion term as an explicit finite difference.

The explicit finite-difference form of equation 8 becomes

$$C_i^n = C_i^o + \frac{D_{x\,i}}{U_i^2 \Delta t}(C_{i+1}^o - C_i^o) - \frac{D_{x\,i-1}}{U_{i-1}^2 \Delta t}(C_i^o - C_{i-1}^o) \tag{9}$$

where C_i^n and C_i^o are concentrations of the parcel i at the beginning and end of the time step, respectively. The finite-difference solution is constructed by adding a new parcel at the upstream boundary at each time step and tracking each parcel as it traverses the system. The parcels are numbered in a downstream direction.

A dimensionless ratio called the dispersion factor, D_f, is defined as

$$D_f = \frac{D_x}{U^2 \Delta t} = \left[\frac{(D_x \Delta t)^{1/2}}{U \Delta t}\right]^2 \tag{10}$$

During the time step, the change in concentration due to dispersion between the parcel and adjacent parcels, ΔC_d is computed as

$$\Delta C_d = D_f \Delta C_x \tag{11}$$

where ΔC_x is the concentration difference between adjacent parcels at the previous time step.

The one-dimensional convective-diffusion equation is also a simplification of an extremely complex phenomena, similar to the case for the unsteady flow equations. The applicability of this equation is governed by several underlying assumptions. Specifically, these assumptions are:

1. The constituent or dispersant is neutrally buoyant or in the solution form.

2. Fickian diffusion theory can be used to describe the longitudinal dispersion process in open channels.

3. The constituent or dispersant moves at the same speed as the fluid parcels.

MODEL IMPLEMENTATION AND RESULTS

Implementation and use of the flow and transport models requires that a series of Eulerian (fixed in space) computational points be selected along longitudinal axis of the water course. Even when using a Lagrangian reference frame, an Eulerain reference frame must be selected to superimpose the Lagrangian solution. The reason for this is because field data were collected using an Eulerian reference frame.

Schematization

The study reach was represented by 34 computational points in the schematic diagrams shown in figures 5 through 8. In a system as large as the lower Mississippi River, the features of a segment must be defined in such a way that the volume of water within the study reach will be correctly represented, and wave propagation speed, which depends principally on top width, will not be biased. With these guidelines in mind, computational points were selected at about 10-mile intervals and reach-averaged cross-sectional properties were computed.

The complete study reach was subdivided into four reaches for flow simulations. This subdivision made data management practical and efficient. However, the study reach was not subdivided for transport simulations. The transport model inherently needs to handle less data.

Cross-sectional information was obtained from current records of the Corps, New Orleans District. Cross sections were supplied about every two river miles. Cross-sectional properties of area, top width, wetted perimeter, and hydraulic radius were computed using conventional techniques and plotted as a step function of a stage. An increment of 1.0 ft was used. Reach-averaged cross-sectional properties were computed as

$$\phi = \frac{\sum_{i=1}^{m-1} (\phi_i + \phi_{i+1})(L_{i+1} - L_i)}{2(L_m - L_1)} \qquad (12)$$

where ϕ is the reach-weighted cross-sectional property, ϕ_i, is the cross-sectional property at a distance, L_i, downstream, and m is the total number of cross sections in the reach.

By reach averaging at 10-mile intervals, the hydraulic model compares well within an acceptable error with the prototype data. Plots of cross-sectional area versus longitudinal distance at stages of 10 and 30 ft above the NGVD of 1929 (fig. 9) show the agreement. The area under each curve represents the volume of the channel at the specified stage. Upon simple numerical integration, using the trapezoidal rule, the volumes differ by less than 0.2 percent at 10 ft and less than 2 percent at 30 ft, based on prototype data.

Flow Simulation Results

Solution of the unsteady flow equations requires proper specification of the initial and boundary conditions. Steady-state initial conditions were used to approximate the true initial conditions. These steady-state initial conditions were obtained from preliminary model runs by holding boundary

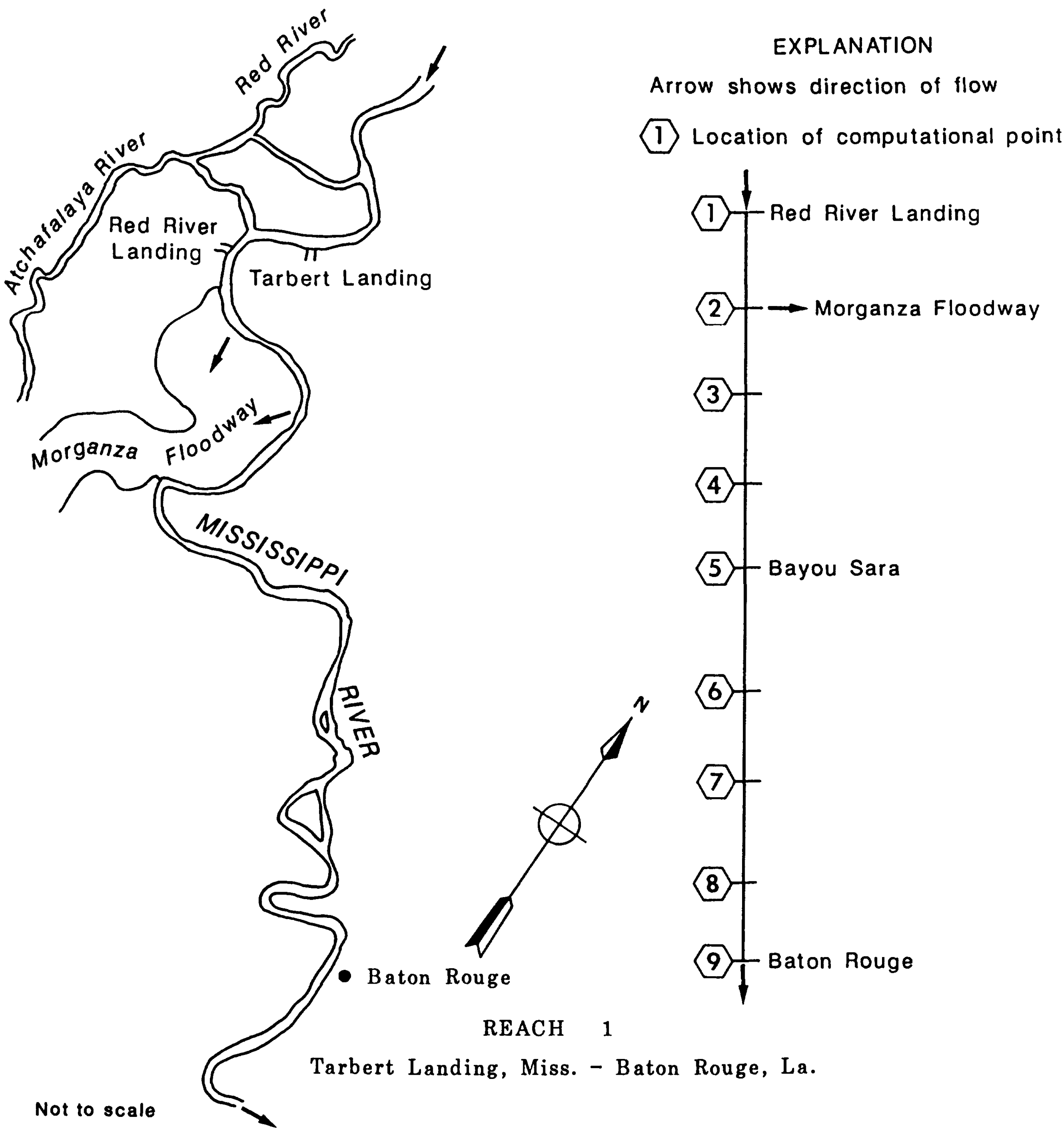


Figure 5.--The schematic of the lower Mississippi River used for flow and transport modeling from Tarbert Landing, Mississippi, to Baton Rouge, Louisiana.

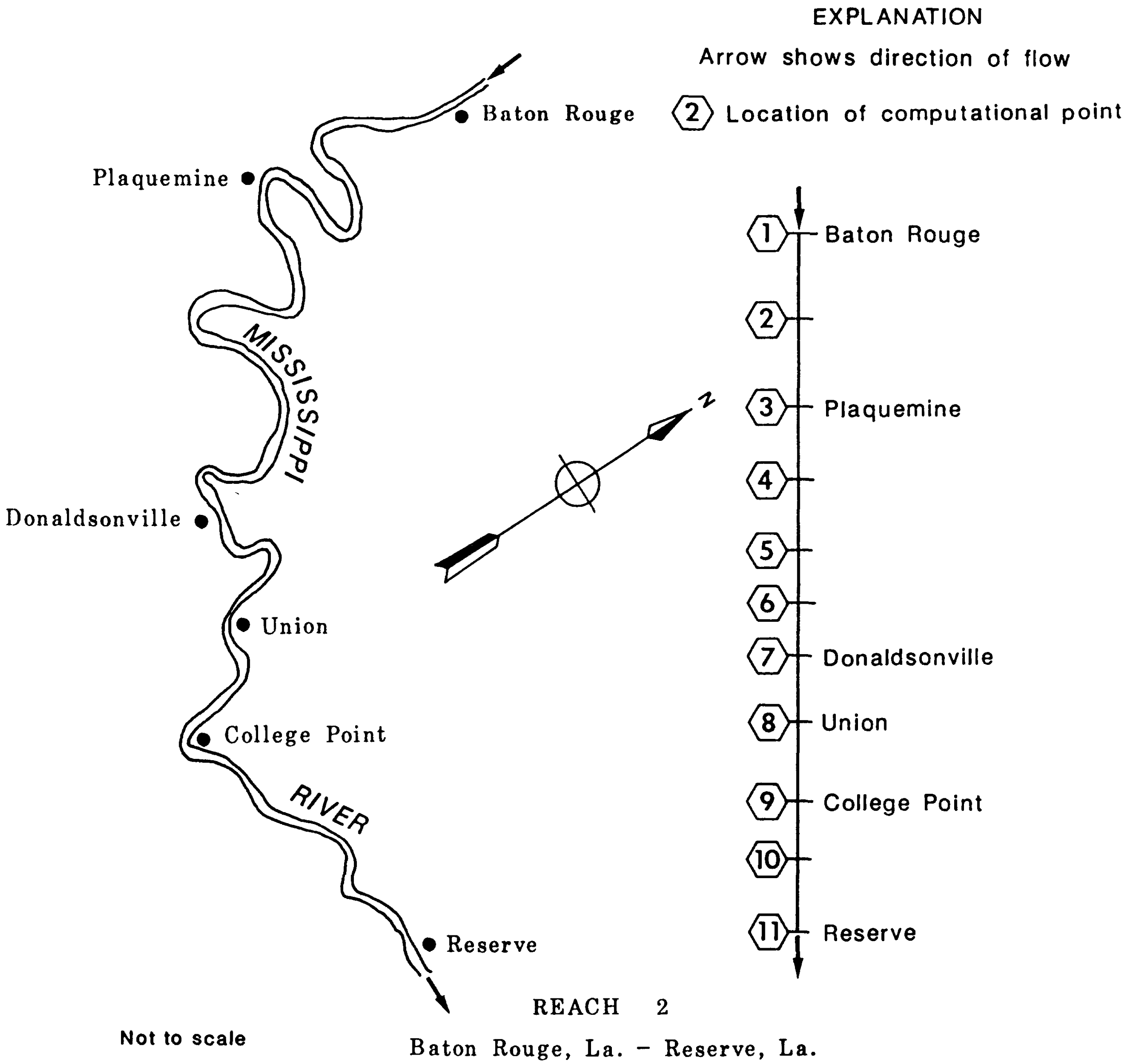


Figure 6.--The schematic of the lower Mississippi River used for flow and transport modeling from Baton Rouge to Reserve, Louisiana.

conditions constant for many time steps. Boundary conditions consisted of water-surface elevations which were recorded daily at 0800 hours (U.S. Army Corps of Engineers, 1980 and 1984). These were input at 24-hour intervals. Intermediate values in time were obtained by linear interpolation in the flow model.

All significant external inflows and outflows of the study reach must also be determined and identified in model implementation. These inflows and outflows are treated as constant flow occurring at a specified computational

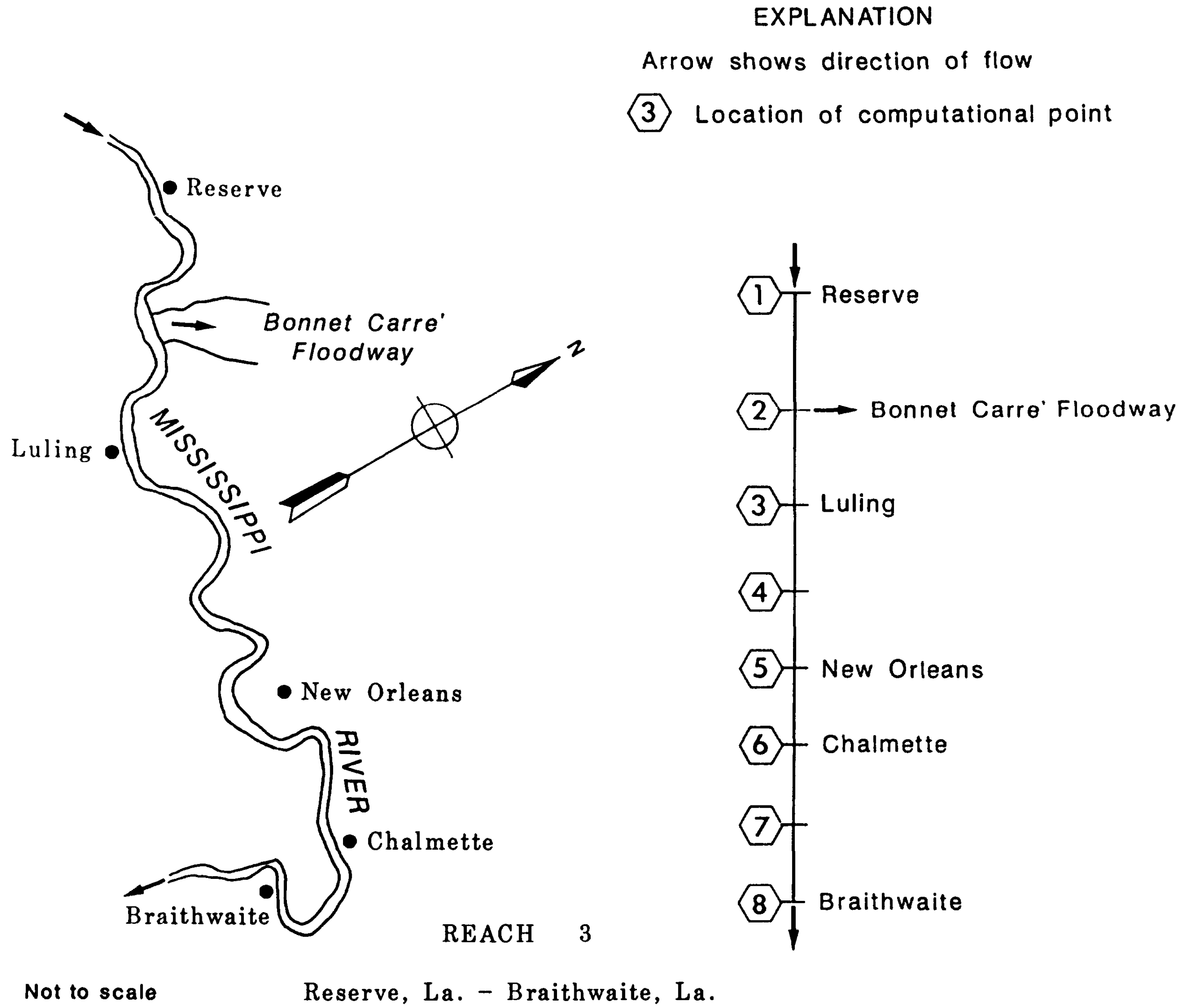


Figure 7.--The schematic of the lower Mississippi River used for flow and transport modeling from Reserve to Braithwaite, Louisiana.

point. There are no significant inflows in the lower Mississippi River system; however, there are two outflows. Distributary flow can occur at the Morganza and Bonnet Carre' Floodways during flood stage. For flow simulations, outflows were time averaged and applied at these two locations at their corresponding computational points.

Flow simulations were conducted using a 3-hour time step. A value of 0.7 was used for both the spatial-derivative weighting factor, θ, and the non-derivative functional weighting factor, χ. A 3-month simulation of the 1979 flood was made to calibrate the model and perform sensitivity analyses. A 3-month simulation of the 1983 flood was made to verify the model and deter-

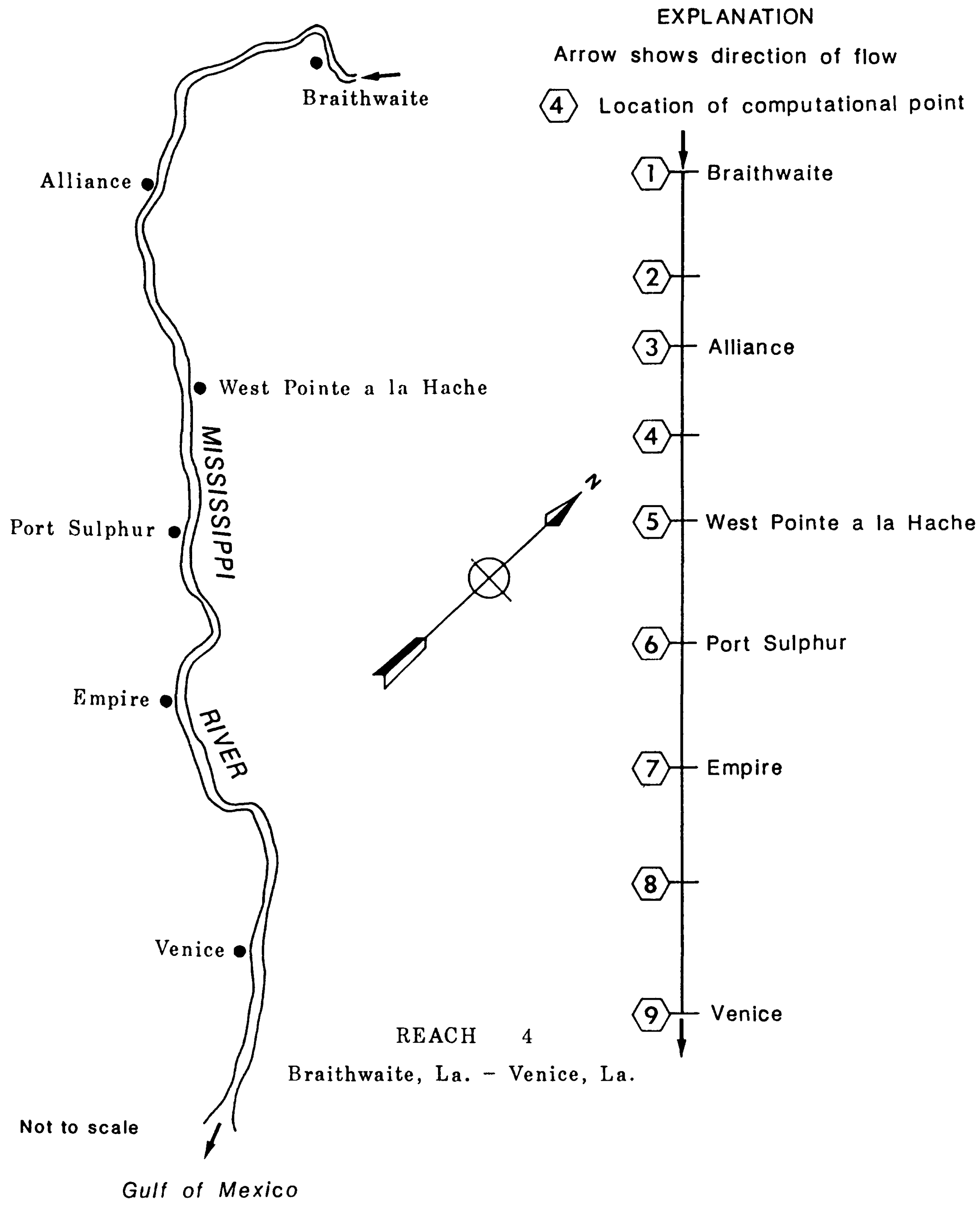


Figure 8.—The schematic of the lower Mississippi River used for flow and transport modeling from Braithwaite to Venice, Louisiana.

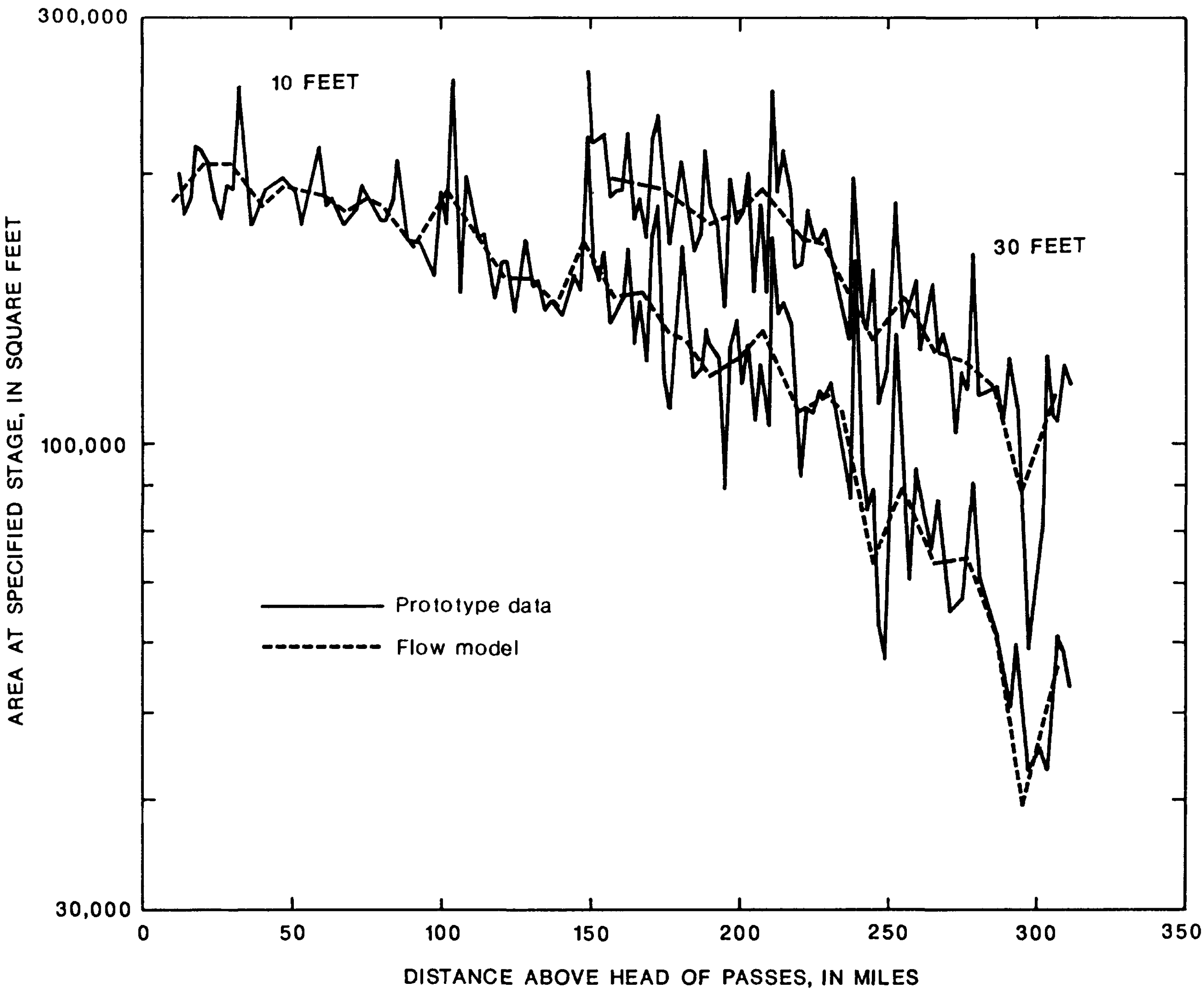


Figure 9.--The comparison between flow model and prototype cross-sectional area at river stages of 10 and 30 feet above the NGVD of 1929.

mine accuracy. Calibration was performed by matching observed and computed stage and discharge using a trial-and-error method. The principal aspect of model calibration was the adjustment of the flow-resistance coefficients.

Gage datum is also an important and critical item that can be adjusted in the calibration process. It especially becomes a very important factor on a mild, almost flat, sloping stream like the lower Mississippi River. In this study all stage records used for modeling purposes were already published or adjusted to the NGVD of 1929. Therefore no adjustment was made in gage datum in the calibration process.

Factors governing flow resistance in an alluvial channel are numerous and complex. The flow resistance of the lower Mississippi River is more complex than described by a uniform-flow formula for rigid-channel boundaries such as the Manning equation (Chow, 1959, p. 99). Nevertheless roughness coefficients (n) were computed from the Manning equation based on hydraulic data collected at Tarbert Landing, Miss.; Baton Rouge, La.; and Belle Chasse, La. (table 2).

Table 2.--Summary of recent hydraulic data collected on the lower Mississippi River at Tarbert Landing, Mississippi; Baton Rouge, Louisiana; and Belle Chasse, Louisiana

[Z is stage, in feet (ft); Q is discharge, in cubic feet per second (ft^3/s); A is cross-sectional area, in square feet (ft^2); U is mean stream velocity, in feet per second (ft/s); B is top width, in feet (ft); S is local water-surface slope, in feet per foot (ft/ft); R is hydraulic radius, in feet (ft); n is Manning's n coefficient computed from Manning equation; T is water temperature, in degree Celsius (°C); IR is Reynolds number; and IF is Froude number]

Date	Z (ft)	$Q*10^{-3}$ (ft^3/s)	$A*10^{-3}$ (ft^2)	U (ft/s)	B (ft)	$S*10^{+5}$ (ft/ft)	R (ft)	n	T (°C)	$IR*10^{-5}$	IF
					Tarbert Landing, Mississippi						
					1982						
12-12	46.15	853	155	5.50	3,500	3.40	44.3	0.0196	10.5	176	0.146
12-20	50.56	1,076	186	5.78	3,900	3.74	47.7	.0207	8.5	188	.147
					1983						
1-24	43.40	718	159	4.51	3,790	3.69	42.0	0.0242	4.5	114	0.123
3-14	40.40	654	144	4.48	3,565	4.11	40.0	.0248	10.5	129	.125
4-25	51.83	1,060	198	5.35	3,900	4.06	50.0	.0240	13.0	207	.133
5-16	54.20	1,280	211	6.06	4,200	3.35	50.0	.0193	21.0	287	.151
5-28	60.33	1,410	233	6.05	4,350	3.35	53.0	.0200	21.0	304	.146
5-31	60.40	1,430	231	6.19	4,200	4.61	55.0	.0236	21.5	327	.147
6- 2	60.40	1,470	235	6.25	4,400	4.44	53.4	.0223	22.0	322	.151
6- 4	60.40	1,470	234	6.25	4,400	4.53	53.2	.0225	22.0	323	.152
6- 7	60.32	1,420	244	5.82	4,500	4.32	54.2	.0240	22.0	305	.139
6- 9	59.90	1,340	228	5.88	4,400	4.02	51.8	.0220	22.5	295	.144
6-11	59.17	1,270	223	5.69	4,300	3.99	51.9	.0227	22.5	285	.139
6-14	57.87	1,160	208	5.57	4,300	4.24	48.4	.0224	23.0	266	.141
6-21	51.58	877	177	4.95	3,800	2.36	46.6	.0187	24.0	232	.128
7-19	32.40	428	105	4.07	3,240	2.59	32.4	.0189	29.5	151	.126
8-15	21.00	237	72.2	3.28	3,230	3.69	22.3	.0218	31.0	87	.122
9-12	17.10	181	63.2	2.85	3,225	3.85	19.6	.0235	29.0	63	.113
10- 3	16.85	193	63.2	3.06	3,200	3.69	19.8	.0215	24.0	61	.121
11-14	20.35	271	77.0	3.50	3,305	3.85	23.3	.0213	14.5	65	.128
12- 5	37.29	674	142	4.70	3,495	4.29	40.6	.0242	10.5	136	.130
12- 6	39.01	737	151	4.87	3,630	4.51	41.6	.0243	10.5	144	.133
					1984						
1- 9	32.99	502	128	3.93	3,364	3.12	38.0	0.0239	1.5	82	0.112
2-14	29.18	447	113	3.47	3,396	3.27	32.3	.0248	6.0	71	.108
3- 7	44.10	817	173	4.73	3,823	1.10	45.3	.0131	8.0	143	.125
4-16	52.41	1,170	221	5.30	4,188	3.53	52.8	.0232	13.5	216	.129

4-30	51.98	1,030	214	4.82	4,170	5.68	51.3	.0319	16.0	206	.119
6-26	35.50	504	121	4.17	3,352	3.53	36.1	.0232	28.0	168	.122
7-19	35.66	524	121	4.33	3,370	3.56	35.9	.0218	28.0	169	.127
9- 4	18.60	222	75.6	2.94	3,580	3.37	21.1	.0224	28.5	70	.113
10-15	18.62	222	73.4	3.02	3,259	3.18	22.5	.0218	22.0	65	.112
11- 5	35.41	532	126	4.22	3,362	3.33	37.5	.0225	18.5	139	.121
12- 3	39.50	664	152	4.37	3,715	3.09	40.9	.0224	9.5	125	.120
					Baton Rouge, Louisiana						
					1973						
4-25	40.48	1,360	198	6.87	2,972	4.24	66.6	0.0231	17.0	394	0.148
5-15	41.48	1,310	174	7.53	3,000	4.70	58.0	.0203	19.5	400	.174
5-22	40.78	1,300	198	6.57	2,990	4.80	66.2	.0257	20.5	408	.142
5-30	39.28	1,200	186	6.45	2,990	4.34	62.2	.0238	22.0	390	.144
6-10	35.68	1,000	171	5.58	2,750	4.08	62.2	.0255	24.0	267	.097
7- 3	29.18	731	150	4.87	2,380	3.88	63.0	.0301	28.5	346	.108
12-27	28.68	771	162	4.76	2,350	4.24	68.9	.0342	7.5	217	.101
					1974						
1-10	31.68	921	174	5.29	2,670	3.73	65.2	0.0278	5.0	211	0.115
1-24	34.58	1,070	188	5.69	2,680	4.03	70.1	.0282	5.5	248	.120
3- 7	32.08	868	160	5.42	2,530	3.83	63.2	.0269	12.0	258	.120
3-21	30.28	812	150	5.41	2,486	3.73	60.3	.0258	15.0	266	.123
3-28	32.98	901	171	5.27	2,905	3.57	58.9	.0255	12.0	233	.121
4-26	30.48	792	157	5.04	2,846	3.62	55.2	.0257	17.0	239	.120
					1975						
1-28	27.14	725	152	4.77	2,390	3.54	63.8	0.0296	8.0	204	0.105
					Belle Chasse, Louisiana						
					1980						
1-10	6.68	606	169	3.59	2,382	1.51	71.0	0.0276	7.5	167	0.075
2- 7	5.11	499	164	3.04	2,312	1.30	70.9	.0302	5.0	131	.064
3- 4	3.73	436	164	2.65	2,251	.99	73.2	.0309	7.0	125	.055
5- 5	9.50	810	173	4.68	2,291	2.35	75.5	.0275	17.5	304	.095
6-18	4.94	498	160	3.11	2,309	1.17	69.3	.0275	27.0	733	.066
7- 9	3.09	397	158	2.51	2,247	.48	70.3	.0221	30.5	205	.053
8- 8	2.09	179	139	1.27	2,100	.31	66.2	.0338	30.0	97	.028
9- 9	2.21	236	155	1.52	2,200	.50	70.5	.0374	29.0	127	.032
10-15	1.85	213	154	1.38	2,260	.40	68.1	.0359	22.0	91	.030
11- 6	2.72	188	151	1.25	2,174	.28	69.5	.0340	18.5	77	.026
12- 9	2.13	338	171	1.98	2,310	.66	74.0	.0341	11.5	108	.041

Table 2.--Summary of recent hydraulic data collected on the lower Mississippi River at Tarbert Landing, Mississippi; Baton Rouge, Louisiana; and Belle Chasse, Louisiana--Continued

Date	Z (ft)	$Q*10^{-3}$ (ft^3/s)	$A*10^{-3}$ (ft^2)	U (ft/s)	B (ft)	$S*10^{+5}$ (ft/ft)	R (ft)	n	T (°C)	$IR*10^{-5}$	IF
					1981						
1-22	0.87	205	162	1.27	2,149	0.34	75.4	0.0387	6.5	61	0.026
2- 2	1.96	204	165	1.24	2,242	.37	73.6	.0404	6.5	58	.026
3-10	4.65	609	161	3.78	2,132	1.14	75.5	.0237	9.5	199	.077
5- 9	3.30	419	165	2.54	2,226	.74	74.1	.0279	21.0	177	.052
6- 3	7.19	757	178	4.25	2,392	1.77	74.4	.0260	21.0	298	.087
7-13	5.59	530	160	3.31	2,157	1.06	74.2	.0258	28.5	274	.068
8- 4	3.80	432	154	2.81	2,142	.81	71.9	.0262	31.0	238	.058
9- 1	2.27	294	160	1.84	2,242	.42	71.4	.0285	29.0	148	.038
10- 7	2.01	184	155	1.19	2,232	.23	69.4	.0322	25.5	86	.025
11- 5	2.71	318	154	2.06	2,205	.45	69.8	.0259	17.0	122	.043
12- 9	1.42	270	156	1.73	2,270	.54	68.7	.0325	13.0	91	.037
					1982						
1-19	4.21	492	153	3.22	2,152	1.21	71.1	0.0276	7.0	148	0.067
2-10	8.66	886	170	5.21	2,382	2.19	71.4	.0226	8.0	248	.109
3-17	7.56	575	135	4.26	2,392	1.81	56.4	.0218	11.0	175	.100
4- 7	9.55	931	186	5.00	2,617	2.35	77.1	.0247	13.5	276	.104
5-12	4.40	411	138	2.98	2,272	1.05	60.7	.0250	26.5	194	.067
6- 8	6.47	580	162	3.58	2,272	1.40	71.3	.0267	27.0	276	.075
7-14	5.26	506	171	2.96	2,342	1.11	73.0	.0292	27.0	234	.061
8- 3	4.01	417	153	2.73	2,210	.82	69.2	.0263	29.5	216	.058
9- 9	3.55	300	155	1.94	2,282	.53	67.9	.0295	30.0	152	.042
10- 5	2.76	234	136	1.72	2.219	.50	61.3	.0302	24.0	106	.039
11-17	2.22	284	155	1.83	2,220	.66	69.8	.0352	18.0	111	.039
12- 7	7.21	729	165	4.42	2,300	1.61	71.7	.0233	13.5	247	.092
					1983						
1- 5	11.09	998	175	5.70	2,460	2.79	71.1	0.0236	10.0	288	0.119
1-28	7.51	721	166	4.34	2,200	1.86	75.5	.0263	5.5	202	.088
3-18	8.27	726	174	4.17	2,206	1.83	78.9	.0280	12.0	245	.083
4-28	10.79	1,070	199	5.38	2,478	2.66	80.3	.0265	14.0	338	.106
5-20	12.22	1,250	208	6.00	2,550	3.01	81.6	.0255	20.0	449	.117
6-24	9.86	855	171	5.00	2,430	2.30	70.4	.0243	25.0	363	.105
7-21	4.48	458	151	3.03	2,180	.85	69.3	.0241	30.0	242	.064
9-14	2.20	245	157	1.56	2,150	.31	73.0	.0294	32.0	137	.032
10- 5	2.31	197	157	1.25	2,115	.30	74.2	.0359	28.0	103	.026
11-17	1.86	280	161	1.74	2,204	.53	73.0	.0345	26.0	134	.036
12- 8	7.41	764	176	4.34	2,224	1.81	77.4	.0214	11.0	244	.087

Data were not available for the computation of the energy slope; therefore, the water-surface slope was substituted. In general, for open-channel flow the water-surface slope can be assumed to approximate the energy slope very well. The slope computations were made by determining the difference in water-surface elevation between the local gage and the next nearest gage (pl. 1).

The computed n values were plotted against the corresponding water discharges on log-linear scales. A linear curve of the form $n = b + m \log(Q)$, where m is the slope, Q is the discharge in cubic foot per second, and b is the intercept at Q less than 1, was fitted to the data using the method of least squares. These equations were then programmed into the computer model in subroutine ETA. When this subroutine was called to compute a flow resistance coefficient, the discharge computed from the previous iteration was used in the equation to approximate the flow resistance coefficient. Figure 10 shows these relations. The data show that n decreases with increasing discharge and varies over a log cycle of discharge at Baton Rouge and Belle Chasse, La. The values of n range from a high of 0.0319 to a low of 0.0131 at Tarbert Landing, Miss., with no apparent pattern in the data for this site. No consistent relations were found either from plots of n against rising and falling stage, water temperature, Reynolds number, or Froude number. Any change that occurs in Manning's n probably occurs as a result of changes in bed forms (dunes to plane), changes in depth on the same bed form (relative roughness), or changes in other physical characteristics such as temperature or sediment load. In order to evaluate the changes in the roughness coefficient, it would be necessary to make a detailed study of the changes in slope, velocities, and cross-sectional geomorphic parameters of the river.

Comparisons between observed and computed stage and discharge at selected gaging stations are shown in figures 11 through 14 for the calibration data of the flood of 1979. The range in discharge was approximately 400,000-1,400,000 ft^3/s in the upper reaches for this event. The range in stage was roughly 20 ft. Computed trends in stage and discharge are shown to follow observed trends for the stations shown. Stations downstream of New Orleans, La., are affected by tidal fluctuations and movement of the saltwater front.

Tidal oscillations in the Gulf of Mexico are propagated into the extreme southern part of the study reach. The observed tides consist of several semidiurnal and diurnal partial tides. Periods of the major partial tides vary from 12.42 to 25.82 hours. Stage data used for boundary conditions and for calibration and verification were collected only once a day and precisely at the same time each day. Intermediate values of stage, necessary for boundary conditions, are linearly interpolated between observed daily values. Therefore, these intermediate values of stage will not represent the tidal fluctuations as well as might be expected if shorter time-intervals were used to collect the stage data. Errors, introduced by imprecise boundary values at intermediate times, partially account for the lack of fit between observed and computed stage in figures 11 through 14. Based on overall results, the model was judged to be calibrated. Accuracies of the simulated stage hydrographs are considered good with root-mean-square errors generally less than 0.4 ft (table 3). Accuracies of the simulated discharge hydrographs also are considered good with root-mean-square errors of less than 10 percent.

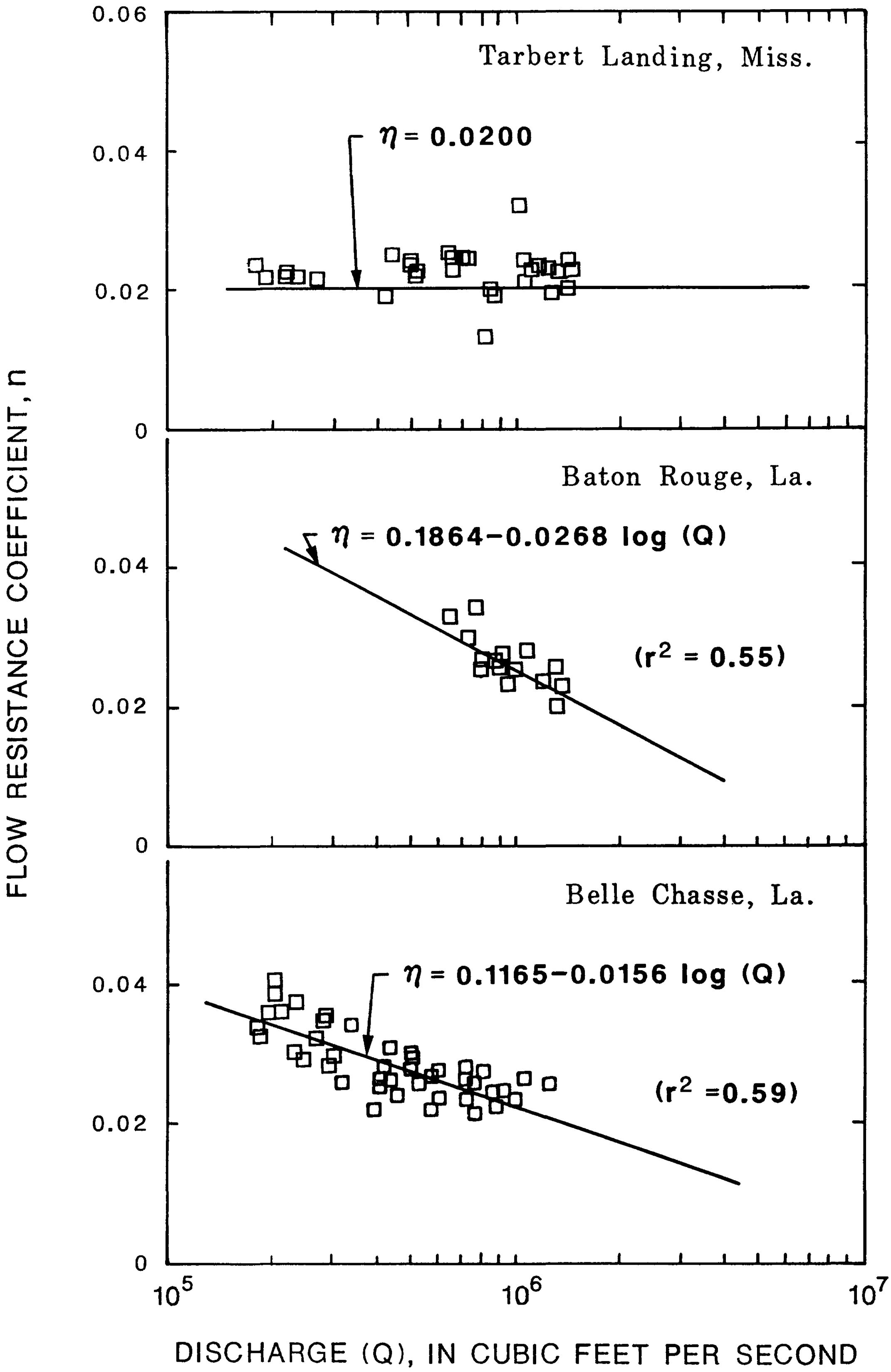


Figure 10.--Relation between flow resistance coefficients and discharge of the lower Mississippi River at selected sites.

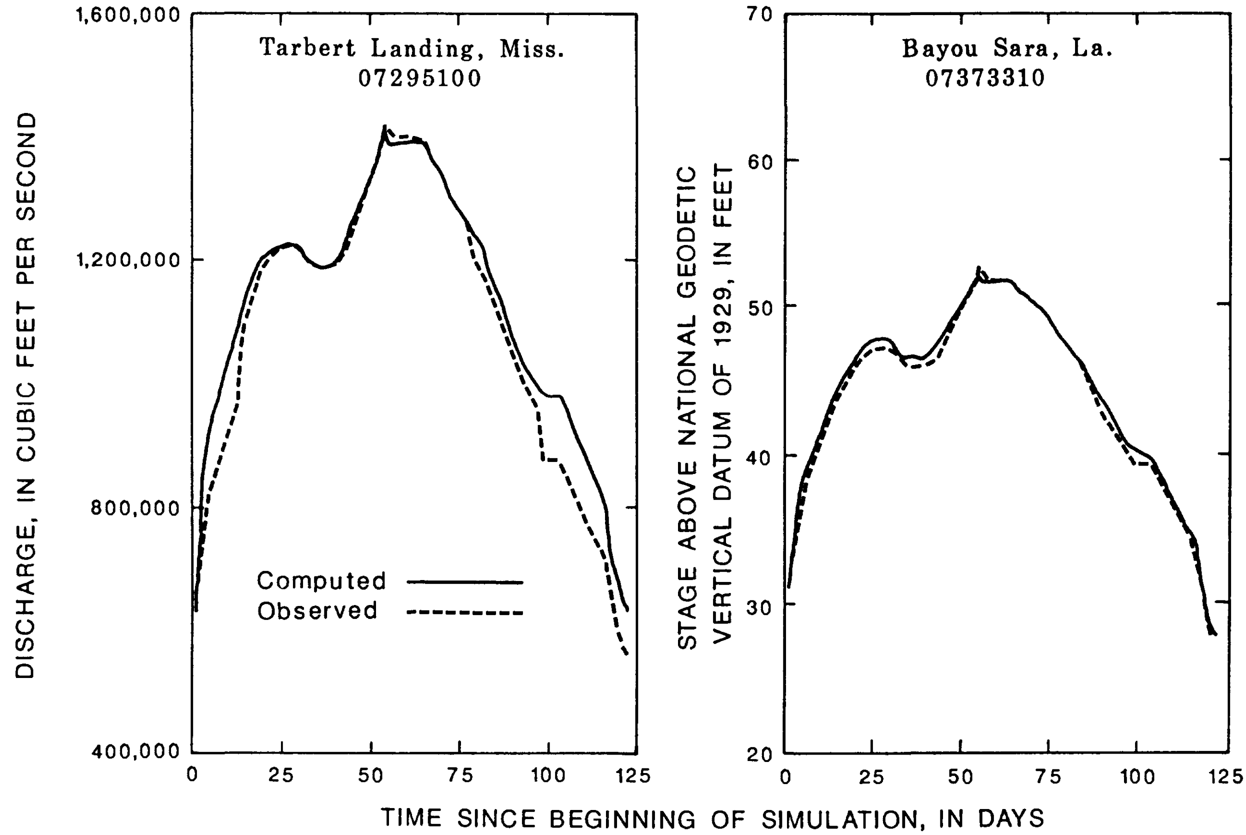


Figure 11.--Comparison of computed and observed discharge and stage of the 1979 flood for the lower Mississippi River at Tarbert Landing, Mississippi, and Bayou Sara, Louisiana.

Comparisons between observed, computed stage, and discharge for the verification data of the flood of 1983 are shown in figures 15 through 18. The range in stage and discharge was similar to that of the 1979 flood used for calibration. The root-mean-square errors are listed in table 3. These results demonstrate the model is an accurate predictor of stage and discharge.

Transport Simulation Results

The transport equation was solved to simulate the convection and dispersion of a conservative, neutrally buoyant substance (solute) in the lower Mississippi River. Data from two previous studies of Stewart (1967) and Martens and others (1974) were used to calibrate and verify the model, respectively. In those studies, a fluorescent tracer (rhodamine BA or WT) was injected into the river at steady flow at Baton Rouge, La. At several downstream locations, the passage of the dye cloud was monitored to determine the transport characteristics at the particular flow condition. Once calibrated and verified, the flow and transport models can be used to describe transport characteristics for additional flow conditions.

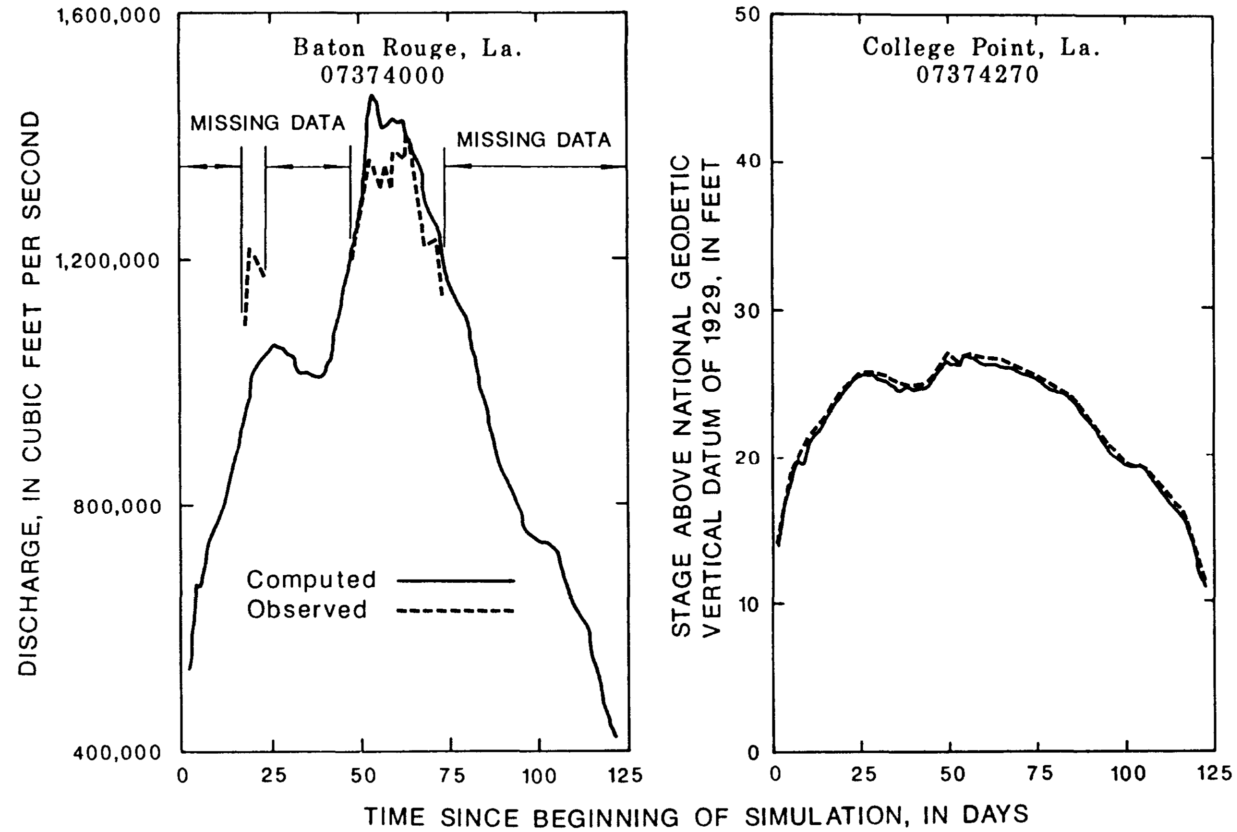


Figure 12.—Comparison of computed and observed discharge and stage of the 1979 flood for the lower Mississippi River at Baton Rouge and College Point, Louisiana.

Solution of the transport equation requires proper specification of initial and boundary conditions. All initial dye concentrations were zero. The boundary conditions were the known values of dye concentrations in time at one of the first sampling locations downstream from the injection point. Even though dye was slug injected at center channel, the measured time-concentration curve at a point downstream was used for the boundary condition because the dye was not well-mixed laterally for some distance downstream. In the model, slug injections are assumed to be well mixed in one time step.

Transport simulations were conducted using a 0.5 hour time step. This short time step was necessary from a practical standpoint in order to adequately resolve the sharp peak and short duration of the measured time-concentration data used for boundary conditions. Hydraulic data necessary for transport calculations were computed using the flow model. Calibration of the transport model was performed by matching observed and computed dye concentrations. The principal aspects of model calibration was the determination of the dispersion factors. Dispersion factors are assigned at each Eulerian computational point; therefore, dispersion characteristics can be changed longitudinally.

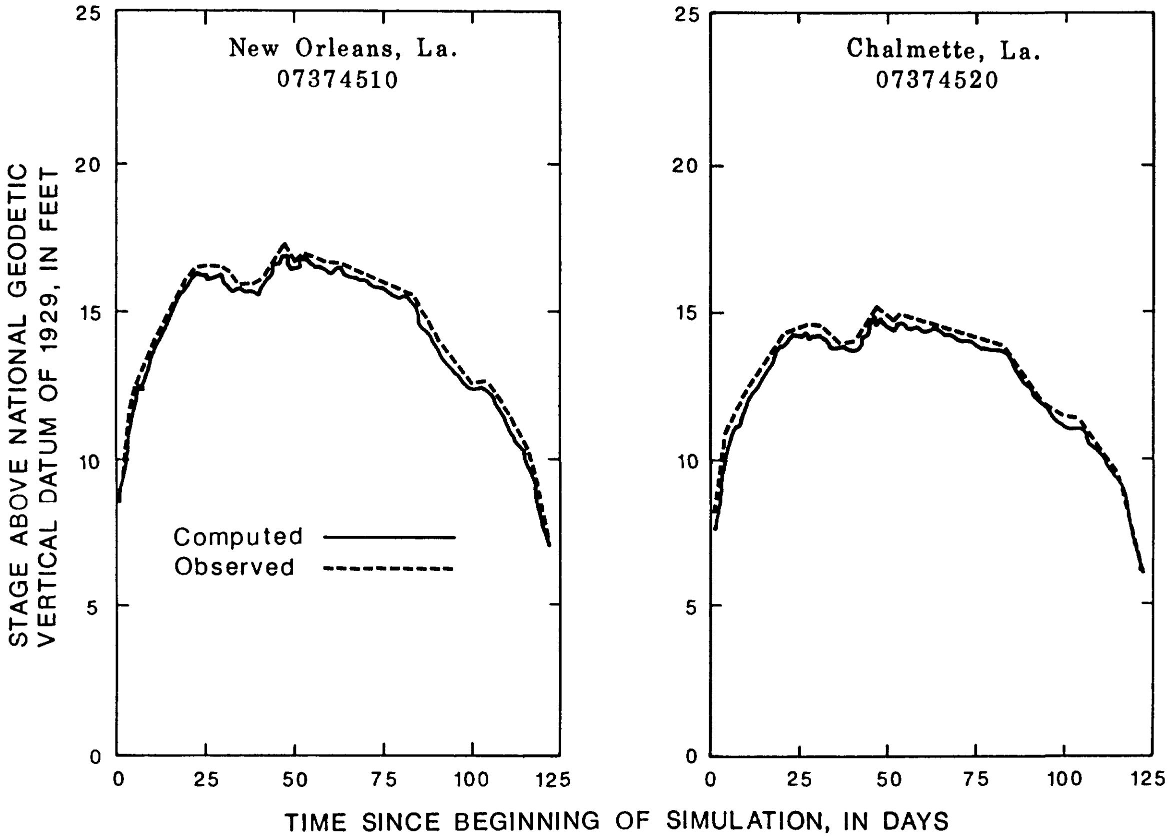


Figure 13.--Comparison of computed and observed stage of the 1979 flood for the lower Mississippi River at New Orleans and Chalmette, Louisiana.

The transport model was calibrated with data collected by Stewart (1967). Rhodamine BA dye was slug injected in the lower Mississippi River at Baton Rouge, La., on September 15, 1965, at 2030 hours. Observed dye concentrations downstream from Baton Rouge at Plaquemine, La., were used for boundary conditions. The passage of the dye cloud was monitored at Union, La. (Sunshine Bridge, No. 07374220), Reserve, La. (No. 07374320), and New Orleans, La. (No. 07374510). A constant dispersion factor, D_f = 0.2, was applied for calibration throughout the entire study reach.

Overall the model does a very good job of simulating the observed dispersion process (fig. 19) and on this basis is accepted as calibrated. Downstream variation of the dispersion factor does not appear to be warranted. The model simulates the reduction in peak dye concentration with distance downstream. The peak dye concentration at New Orleans, was 38 percent of the peak dye concentration at Plaquemine, even though it was tracked for over 110 hours. The trailing-edge tails are not matched closely, but durations of the time-concentration profiles are in close agreement. The average root-mean-square error associated with the calibration was 0.18 ppb (table 3).

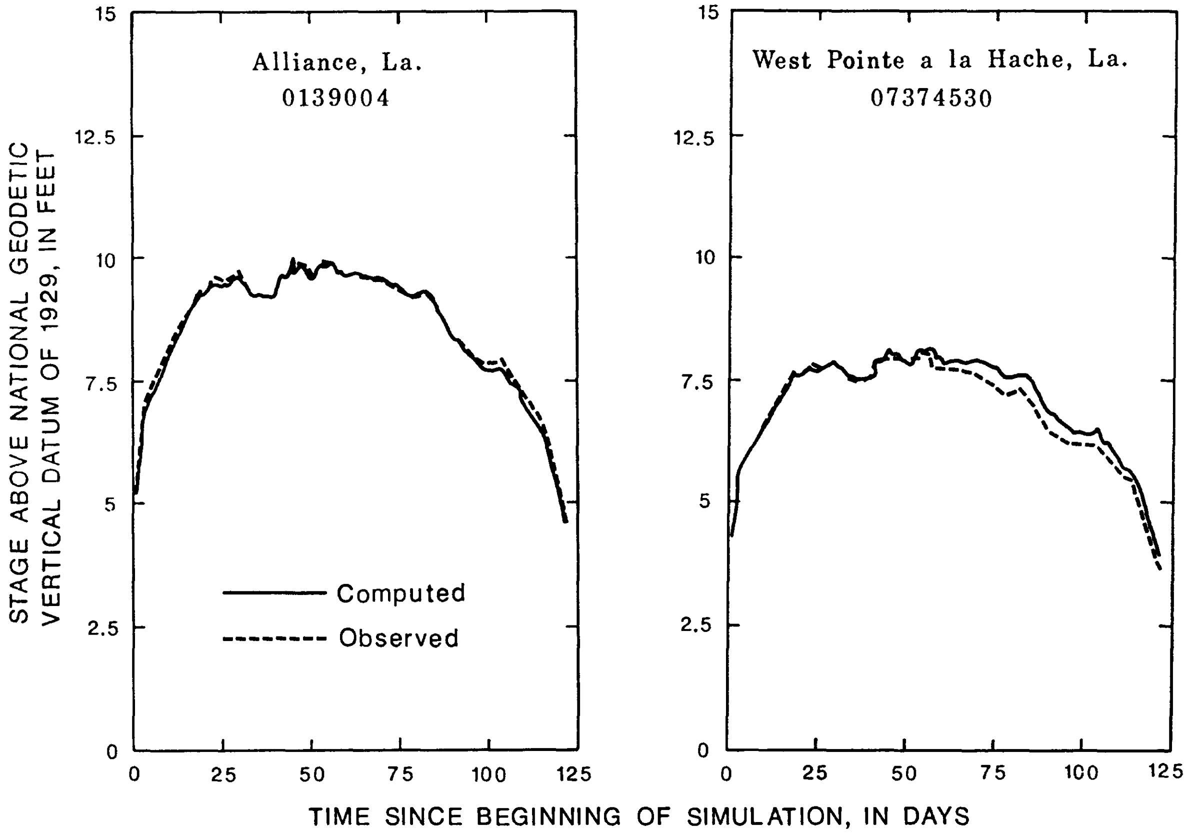


Figure 14.—Comparison of computed and observed stage of the 1979 flood for the lower Mississippi River at Alliance and West Pointe a la Hache, Louisiana.

The dispersion factor, D_f, can be converted to a dispersion coefficient, D_x, by use of equation 10. The mean velocity in the reach is 2.11 ft/s, so the optimum dispersion coefficient, D_x, was 1,600 ft²/s. The average slope was 0.7213×10^{-5} ft/ft and the average depth was about 59 ft. Fischer (1973) reported observed dispersion coefficients in natural rivers vary from 74 to 7,500 times the product of the depth and shear velocity. The calibrated dispersion coefficient for the lower Mississippi River is approximately equal to 230 times the product, which is a reasonable value relative to those observed.

The transport model was verified with data collected by Martens and others (1974). Rhodamine WT dye was slug injected in the lower Mississippi River again at Baton Rouge, La., on April 4, 1974, at 0900 hours. Observed dye concentrations in time downstream from Baton Rouge at Plaquemine, La., were used for the boundary conditions. The passage of the dye cloud was monitored at the same sites as for the calibration data set and at two additional sites located further downstream at Belle Chasse, La. (No. 07374525), and West Pointe a la Hache, La. (No. 07374530).

Table 3.--*Errors associated with calibration and verification of flow and transport models of the lower Mississippi River*

[ft³/s, cubic feet per second; ft, feet; ppb, parts per billion]

Station name	Quantity	Root-mean-square error
	Flow calibration	
Tarbert Landing, Miss-------	discharge	54,230 ft³/s
Bayou Sara, La-------------	stage	.39 ft
Baton Rouge, La------------	discharge	109,700 ft³/s
Donaldsonville, La----------	stage	.26 ft
College Point, La-----------	stage	.32 ft
New Orleans, La------------	stage	.31 ft
Chalmette, La--------------	stage	.31 ft
Belle Chasse, La-----------	discharge	176,200 ft³/s
Alliance, La---------------	stage	.60 ft
West Pointe a la Hache, La--	stage	1.13 ft
	Transport calibration	
Union (Sunshine Bridge), La-	concentration	0.22 ppb
Reserve, La----------------	concentration	.14 ppb
New Orleans, La------------	concentration	.17 ppb
	Flow verification	
Tarbert Landing, Miss-------	discharge	49,950 ft³/s
Bayou Sara, La-------------	stage	.26 ft
Baton Rouge, La------------	discharge	102,900 ft³/s
Donaldsonville, La----------	stage	.29 ft
College Point, La-----------	stage	.52 ft
New Orleans, La------------	stage	.43 ft
Chalmette, La--------------	stage	.43 ft
Belle Chasse, La-----------	discharge	106,000 ft³/s
Alliance, La---------------	stage	.22 ft
West Pointe a la Hache, La--	stage	.63 ft
	Transport verification	
Union (Sunshine Bridge), La-	concentration	0.13 ppb
Reserve, La----------------	concentration	.097 ppb
New Orleans, La------------	concentration	.067 ppb
Belle Chasse, La-----------	concentration	.067 ppb
West Pointe a la Hache, La--	concentration	.074 ppb

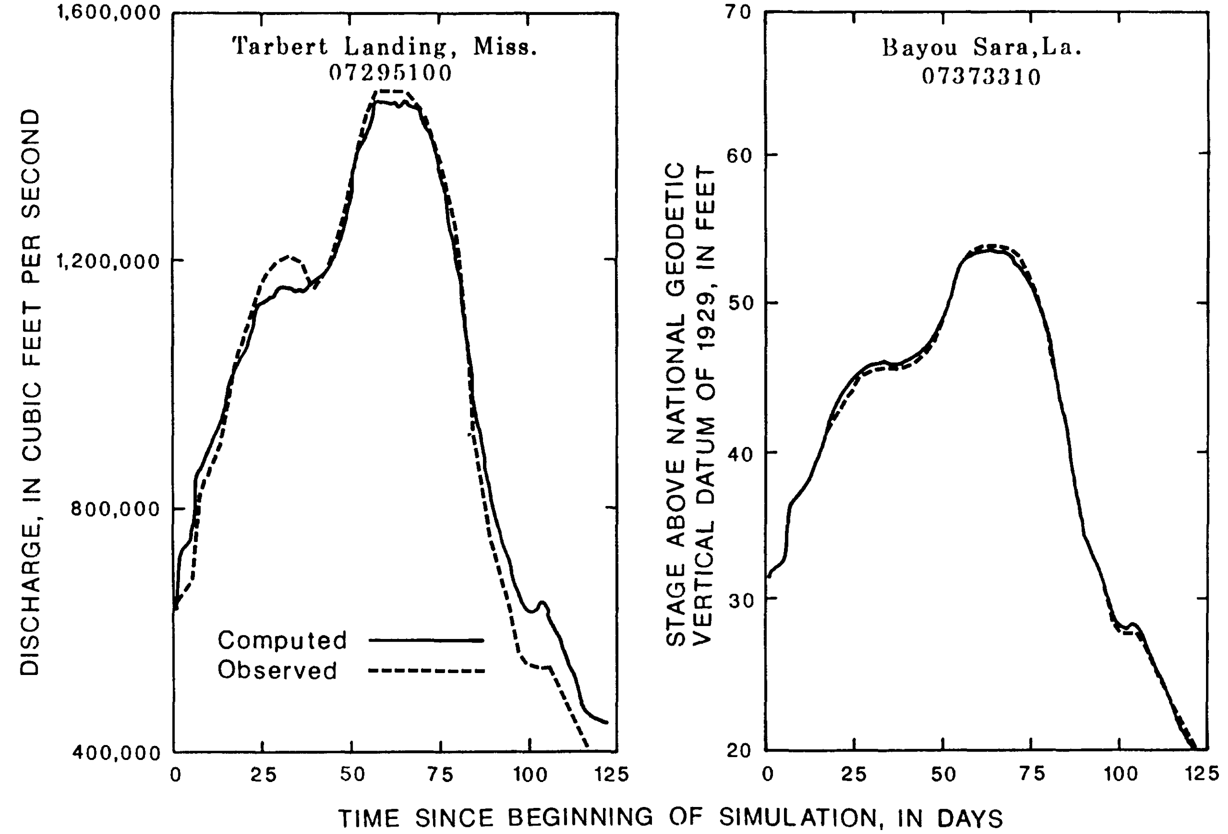


Figure 15.—Comparison of computed and observed discharge and stage of the 1983 flood for the lower Mississippi River at Tarbert Landing, Mississippi, and Bayou Sara, Louisiana.

A comparison of the computed and observed time variations of concentration at each sampling site for verification for the model is shown in figure 20. The comparison of curves shows the model computed time variations of dye concentration correspond closely with observed time variations of dye concentrations. The root-mean-square error for using a dispersion factor of 0.2 from Baton Rouge to West Pointe a la Hache, La., averages 0.087 ppb. Based on these results, a dispersion factor of 0.2 was used for the entire study reach.

Sensitivity Analysis

Numerical experiments were used to perform a sensitivity analysis on controlling parameters of the flow model. The analysis permits model users to determine the extent to which uncertainty in the input parameters results in uncertainty in the predicted hydrodynamics. Five sets of numerical experiments were conducted to investigate: (1) the effects of computational time-step size, (2) variation in flow-resistance coefficients, (3) making a rigid cross-sectional boundary assumption, (4) effects of wind, and (5) errors in

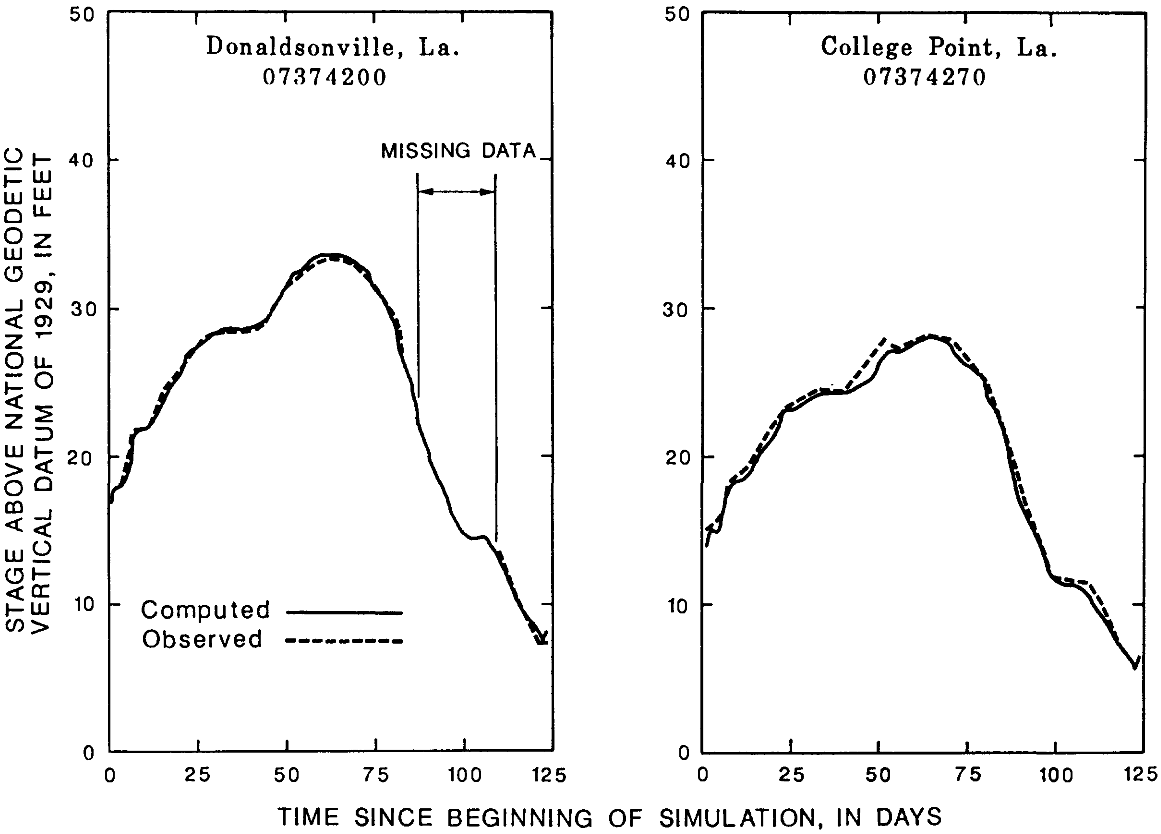


Figure 16.—Comparison of computed and observed stage of the 1983 flood for the lower Mississippi River at Donaldsonville and College Point, Louisiana.

boundary-value data. These experiments were conducted on reach 2, extending from Baton Rouge to Reserve, La. Similar results are expected for other reaches.

The flood of 1979 was routed through reach 2 using different time-step sizes. There was little or no noticeable difference between stage and discharge hydrographs computed with 0.5-, 1-, 3-, 6-, 12-, or 24-hour time steps. However as the time-step size increased beyond 12 hours, significantly more iterations were needed to solve the non-linear flow equations to the closure criteria set at 0.01-foot change in stage and 4,000-ft^3/s change in discharge per iteration.

Knowledge of the effects of the flow-resistance coefficient on the computed stages and discharges is essential for understanding the flow model. If values for n are too small, flow resistance is reduced and discharge and momentum are increased. An excessive value for n decreases discharge and momentum. The computed stages are not significantly affected by the particular values of n that are used in the simulations if they are reasonable.

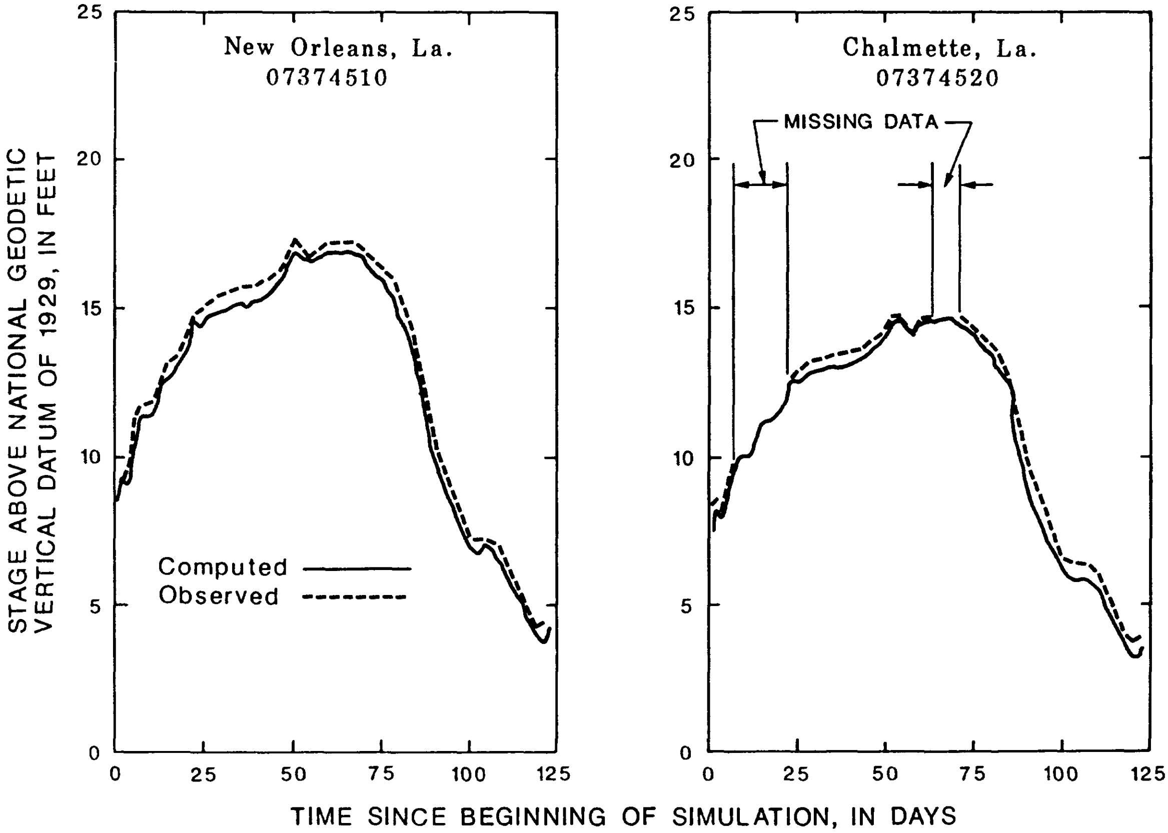


Figure 17.--Comparison of computed and observed stage of the 1983 flood for the lower Mississippi River at New Orleans and Chalmette, Louisiana.

The flow-resistance coefficient was varied with discharge to better reproduce the observed data. The variation of the discharge hydrograph with the assumed η value provides an indication of the precision, a resistance coefficient should be actually defined. Using the 1979 flood and the calibrated η value, the computed discharge hydrograph for College Point, La., is shown in figure 21 as the hydrograph denoted by η_o. Upon increasing the values of η_o by 10 percent and repeating the computations, the resulting discharges are lower than those computed using η_o (fig. 21). The η_o values then were decreased by 10 percent and the computations repeated. The discharge hydrograph computed using the decreased η value is higher than the discharge hydrograph computed using η_o (fig. 21). If an average constant η value were used and the computations repeated, the resulting discharge hydrograph behaves in the manner shown in figure 21 as denoted by η_{ave}.

Levees restrict the lateral movement of the channel of the lower Mississippi River, but the bed is free to adjust itself to the imposed hydraulic conditions and sediment load. Cross sections in the flow model are assumed to be fixed (no scour or fill). This assumption is not completely

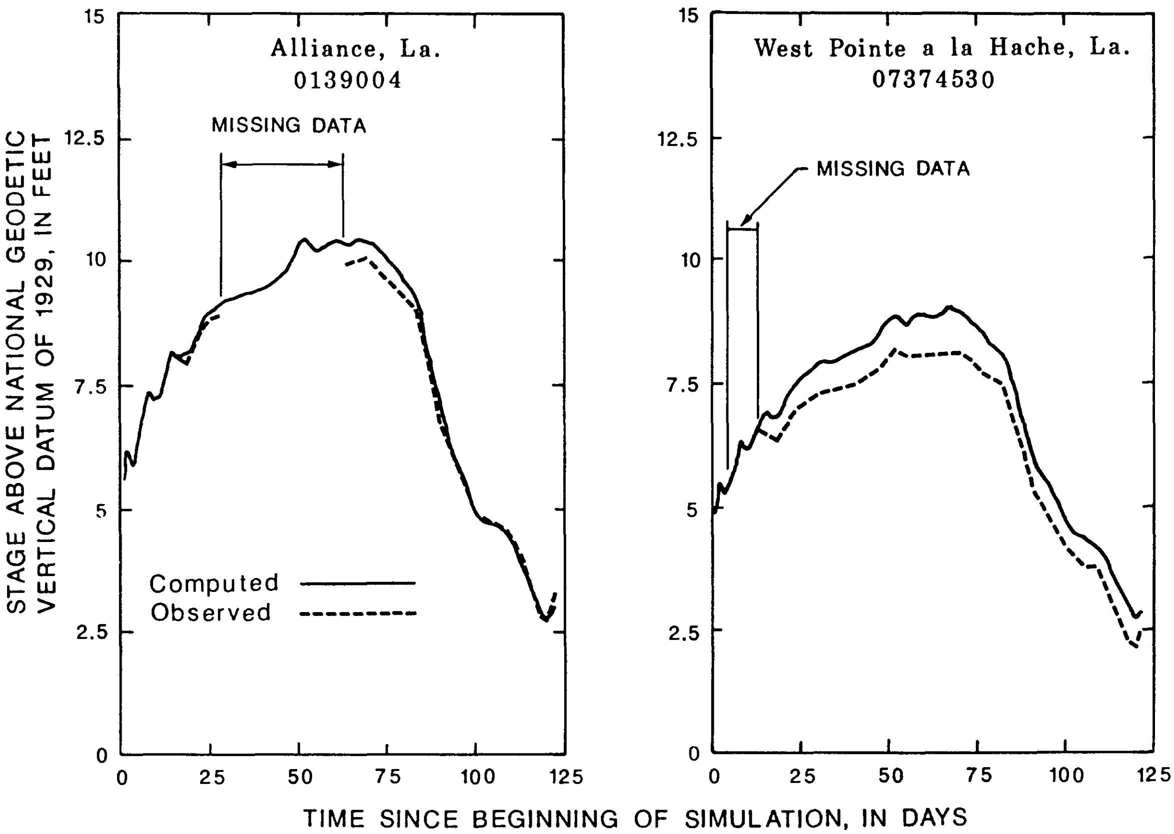


Figure 18.--Comparison of computed and observed stage of the 1983 flood for the lower Mississippi River at Alliance and West Pointe a la Hache, Louisiana.

correct so the effects of varying cross-sectional geometry were explored. The elevation of each point was increased by 10 ft at a single cross section of one submodel and the computations were repeated. The results showed little difference between the two hydrographs at College Point, La. Therefore, the assumption of a fixed cross-sectional boundary seems reasonable. Minor changes in the bed elevations, such as the passage of a dune, do not significantly change the storage characteristics of the channel and hence do not significantly affect the computed stages or discharges.

Wind effect on the stage and discharge hydrographs was also investigated. The last term on the left side of equation 2 and wind data collected at the Baton Rouge Municipal Airport were used for this investigation. Based on model runs with and without wind conditions imposed on the system, average wind conditions were found to have no effect on stage or discharge hydrographs. Increasing the average winds by 100 and 200 percent had little effect on the hydrographs.

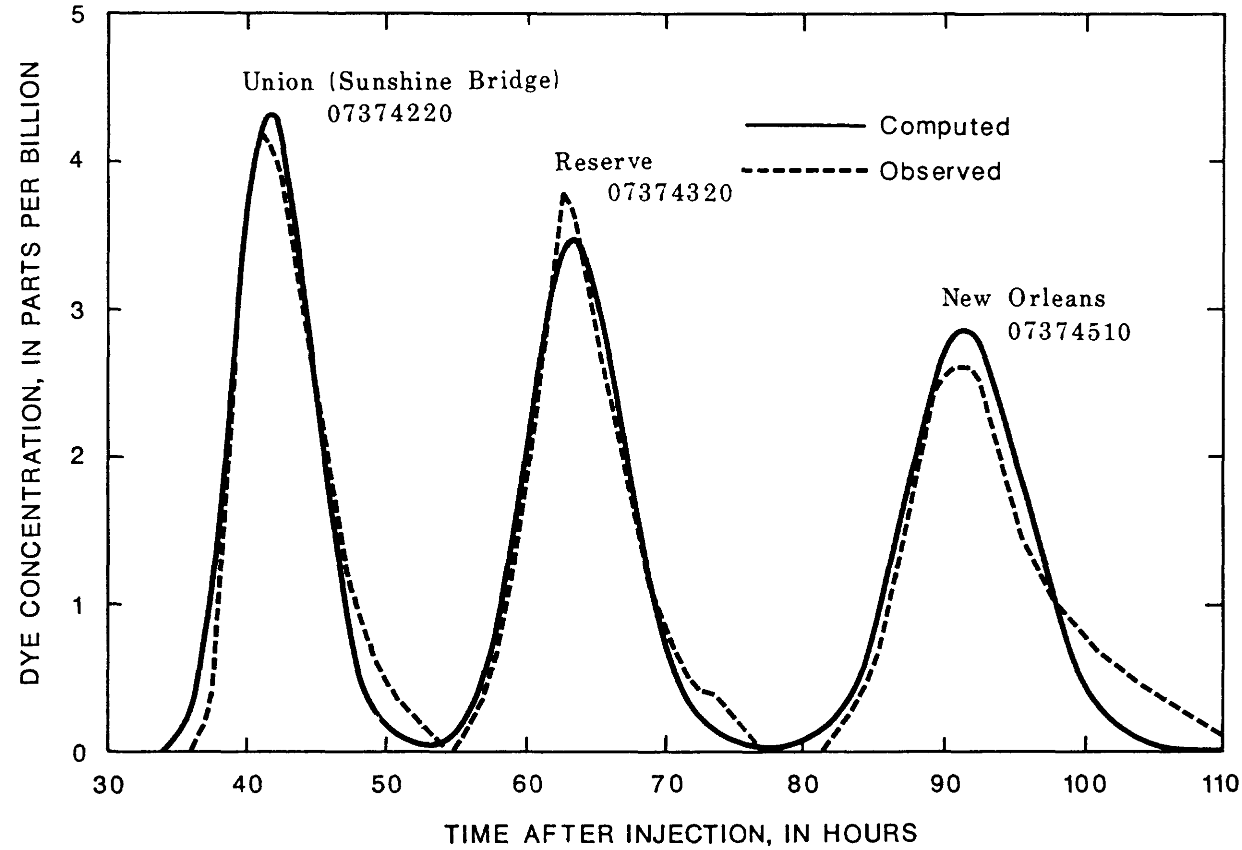


Figure 19.—Comparison of computed and observed time variations of dye concentration in the lower Mississippi River downstream from Baton Rouge, Louisiana, for the September 1965 injection.

Errors in boundary-value data will adversely effect the solution of the flow model. Recorded stages that are used for boundary-value data may sometimes be incorrectly defined because of datum errors, survey inaccuracies, or settling of the gage. The net result of such errors is an increase of a decrease in water-surface slope throughout the channel reach. Increasing the water-surface slope (fall) by 2.0 ft (approximately 5 percent of mean stage) results in just over 10 percent error in discharge. The opposite is true when decreasing the water-surface slope. Therefore, errors in boundary-value data will be magnified in the solution of the flow model.

The transport model is sensitive to the assumed value of the dispersion factor used in the analysis. Figure 22 shows time concentration curves computed with four different dispersion factors. Increasing the dispersion factor by 100 percent from 0.2 to 0.4 results in a peak-concentration error of -14 percent. Decreasing the dispersion factor by 50 percent from 0.2 to 0.1 results in a peak-concentration error of +47 percent.

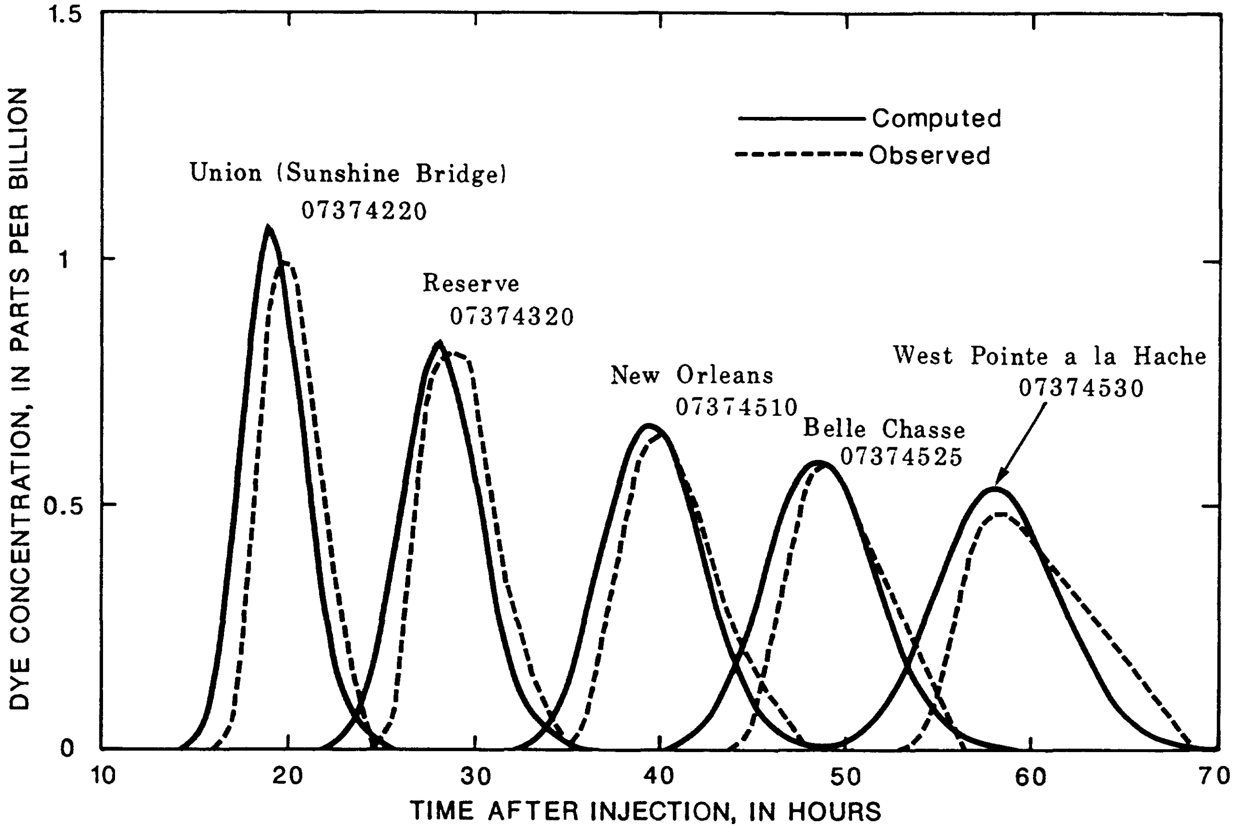


Figure 20.--Comparison of computed and observed time variations of dye concentration in the lower Mississippi River downstream from Baton Rouge, Louisiana, for the April 1974 injection.

APPLICATION OF FLOW AND TRANSPORT MODELS

Because of its size, the flow dynamics of the lower Mississippi River are difficult to measure and analyze. In this regard, mean-daily discharge and solute-transport rates available from these models can provide useful information for assessing the quantity and quality of water available throughout the study reach.

Flow and transport models can also be a useful tool for predicting the transport and fate of water-quality constituents in the lower Mississippi River. These models are applicable for both steady and unsteady flow with variable boundary conditions and loads. Given the reaction kinetics it is possible to simulate several nonconservative water-quality constituents including temperature, algae, ammonia, nitrite, nitrate, orthophosphate, biochemical oxygen demand, dissolved oxygen, and coliform.

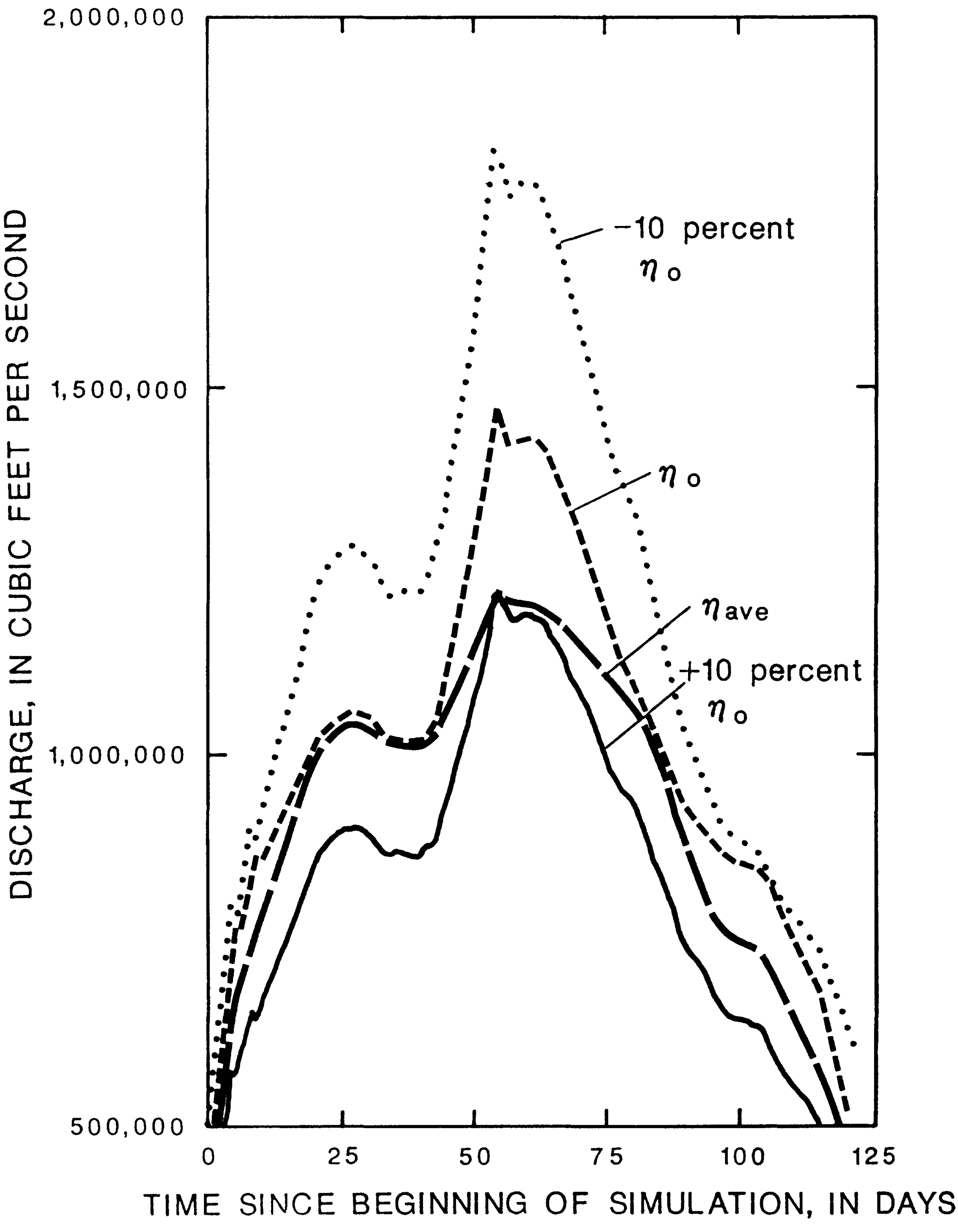


Figure 21.--Discharges computed with different flow-resistance coefficients for the lower Mississippi River at College Point, Louisiana, for the 1979 flood.

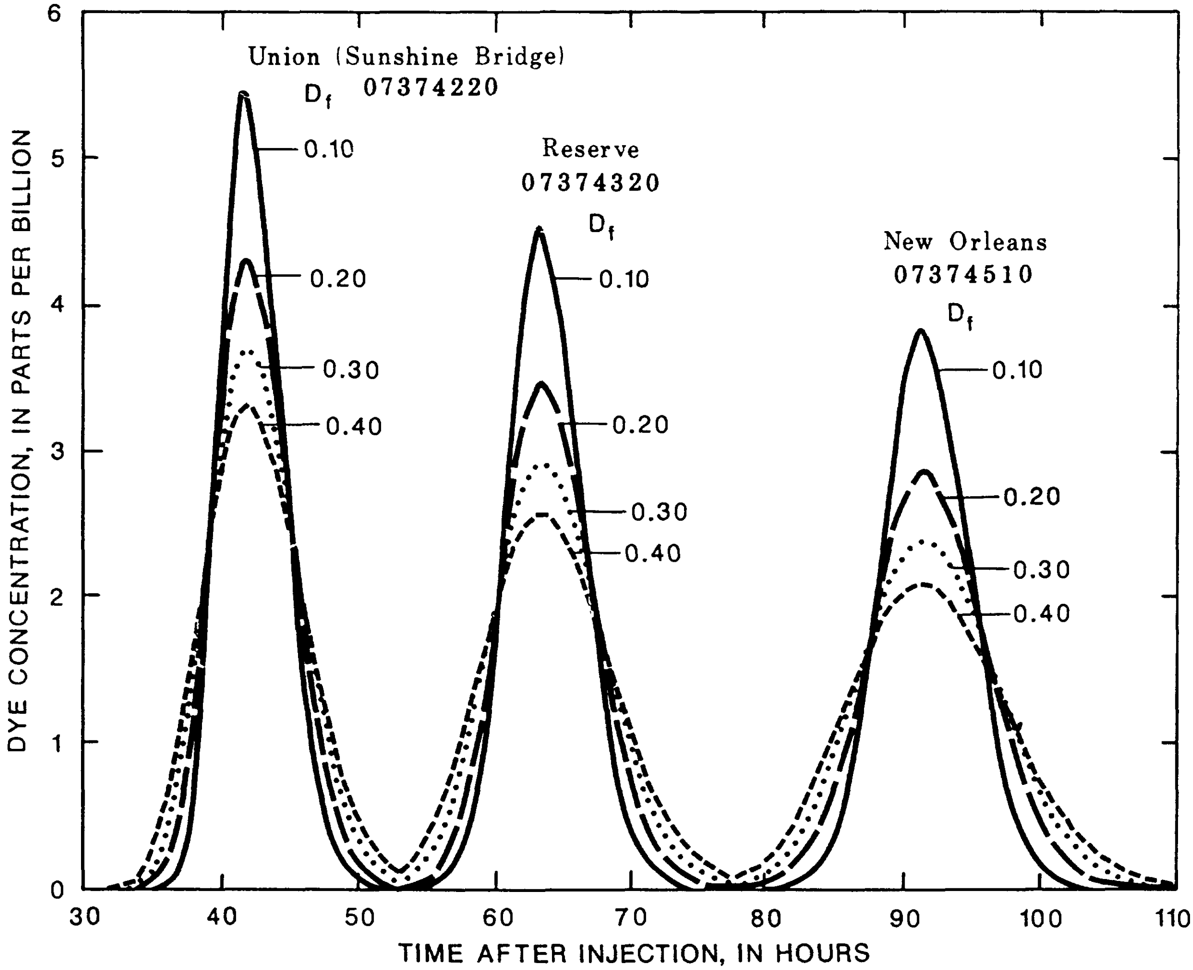


Figure 22.--Time-concentration curves computed with different dispersion factors, D_f, for the lower Mississippi River, for the September 1965 injection.

SUMMARY

Mathematical models of streamflow and solute transport have been developed for a 295-mile reach of the main stem of the lower Mississippi River. The reach extends from Tarbert Landing, Miss., to Venice, La. Both the flow and transport models were calibrated and verified using only historic data. The fully dynamic, one-dimensional equations of unsteady flow and convective diffusion have been solved uncoupled. The flow model uses a weighted, four-point, implicit finite-difference approximation for solution of the flow equations. The functional form of the flow-resistance coefficient was derived from observed data. The transport model uses an explicit finite-difference approximation of the continuity of mass equation for solution in a Lagrangian coordinate system. The study reach was represented by 34 computational points. These computational points were selected at about 10-mile intervals and reach-averaged cross-sectional properties were computed. The study reach was subdivided into four reaches for flow simulations, but was not subdivided for transport simulations.

The flow model was calibrated with 3 months of stage and discharge data from the flood of 1979 and verified with 3 months of stage and discharge data from the flood of 1983. A 3-hour time step was used in the simulations. In simulating an event with a 20-foot change in stage, root-mean-square errors for stage ranged from 0.26 to 1.13 ft and averaged 0.47 ft for calibration. For a discharge ranging from 40,000 to 1,400,000 ft^3/s, root-mean-square errors for discharge ranged from 54,230 to 176,200 ft^3/s and averaged 113,400 ft^3/s for calibration. The root-mean-square errors for stage ranged from 0.22 to 0.63 ft and averaged 0.40 ft for verification. The root-mean-square errors for discharge ranged from 49,950 to 106,000 ft^3/s and averaged 86,280 ft^3/s for verification.

The transport model was calibrated with dye-tracer data from the dye injection of 1965 and verified with dye-tracer data from the dye injection of 1974. Calibration resulted in a range of root mean-square-errors of dye concentration from 0.14 to 0.22 ppb and averaged 0.18 ppb for peak dye concentrations ranging from 3.0 to 4.5 ppb. The verification resulted in a range of root-mean-square errors of dye concentration from 0.067 to 0.13 ppb and averaged 0.087 ppb for peak dye concentrations ranging from 0.5 to 1.0 ppb.

Numerical experiments were used to perform a sensitivity analysis of the controlling parameters of both models. Computed stages and discharges are insensitive to the length of the time step and average wind conditions. The computed stages are insensitive to changes in flow-resistance coefficients, but discharges are highly sensitive to these changes. Error in boundary-value data will adversely effect the solution of the flow model. Changing the water-surface slope by approximately 5 percent results in over 10 percent error in discharge. A rigid boundary assumption for channel cross sections was found to be reasonable. Computed time-concentration curves are extremely sensitive to changes in the dispersion factor.

These flow and transport models provide a convenient and economical framework to further analyze the hydrodynamics and water quality of the lower Mississippi River in Louisiana.

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