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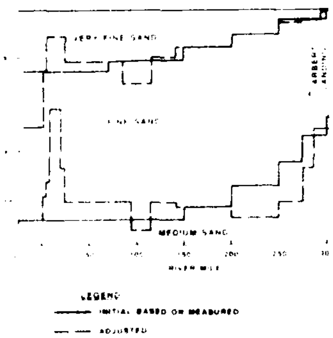

HYDRAULICS
LABORATORY

TECHNICAL REPORT HL-92-6

LOWER MISSISSIPPI RIVER TARBERT LANDING TO EAST JETTY SEDIMENTATION STUDY

Numerical Model Investigation

by

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13. ABSTRACT *(Maximum 200 words)*

A one-dimensional numerical model (TABS-1) of the Mississippi River between Tarbert Landing, at river mile 306, and East Jetty, at river mile -20, was developed. The model was used to evaluate long-term aggradation and degradation trends, the effect of various flow diversion schemes on dredging in Southwest Pass, and washout of a sediment sill at river mile 63, and for preliminary evaluation of dike fields for Redeye Crossing at river mile 224. The numerical model was adjusted to simulate measured degradation and aggradation that occurred between the 1963 and 1975 hydrographic surveys. The model was further adjusted to simulate reported dredging quantities in Southwest Pass.

Circumstantiation of the model was attempted by comparing calculated and measured aggradation between 1975 and 1983, and calculated and measured bed material gradations between 1975 and 1989. The model was also circumstantiated by simulating measured 1989 sediment accumulation in the vicinity of Cubits Gap

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and the 1988-89 washout at the sediment sill at river mile 63.4. Decreases in the supply of sediment from upstream of Tarbert Landing were found to be the primary cause of degradation in this reach of the river. It was also determined that diverting flows may provide a temporary benefit for dredging in Southwest Pass, but in the long run an increase in dredging would occur.

PREFACE

The numerical model investigation of the lower Mississippi River reported herein was conducted at the US Army Engineer Waterways Experiment Station (WES) at the request of the US Army Engineer District, New Orleans (LMN).

The investigation was conducted in three phases during the period February 1984 to April 1991 by personnel of the Hydraulics Laboratory at WES under the direction of Messrs. Henry B. Simmons and Frank A. Herrmann, Jr., Directors of the Hydraulics Laboratory; Richard A. Sager, Assistant Director of the Hydraulics Laboratory; Marden B. Boyd, Chief of the Waterways Division (WD); and Michael J. Trawle, Chief of the Math Modeling Branch (MMG), WD. Mr. William A. Thomas (WD) provided general guidance and review. The project engineer was Mr. Ronald R. Copeland. Authors of this report were Messrs. Copeland and Thomas. Ms. Peggy Hoffman provided technical assistance.

During the course of this study, close working contact was maintained with Messrs. Cecil Soileau, Bill Garrett, Jim Austin, and Tim Axman and Ms. Nancy Powell, LMN, who provided data, technical assistance, and review.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
degrees Fahrenheit	5/9*	degrees Celsius or Kelvins
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
tons (2,000 pounds, mass)	907.1847	kilograms

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

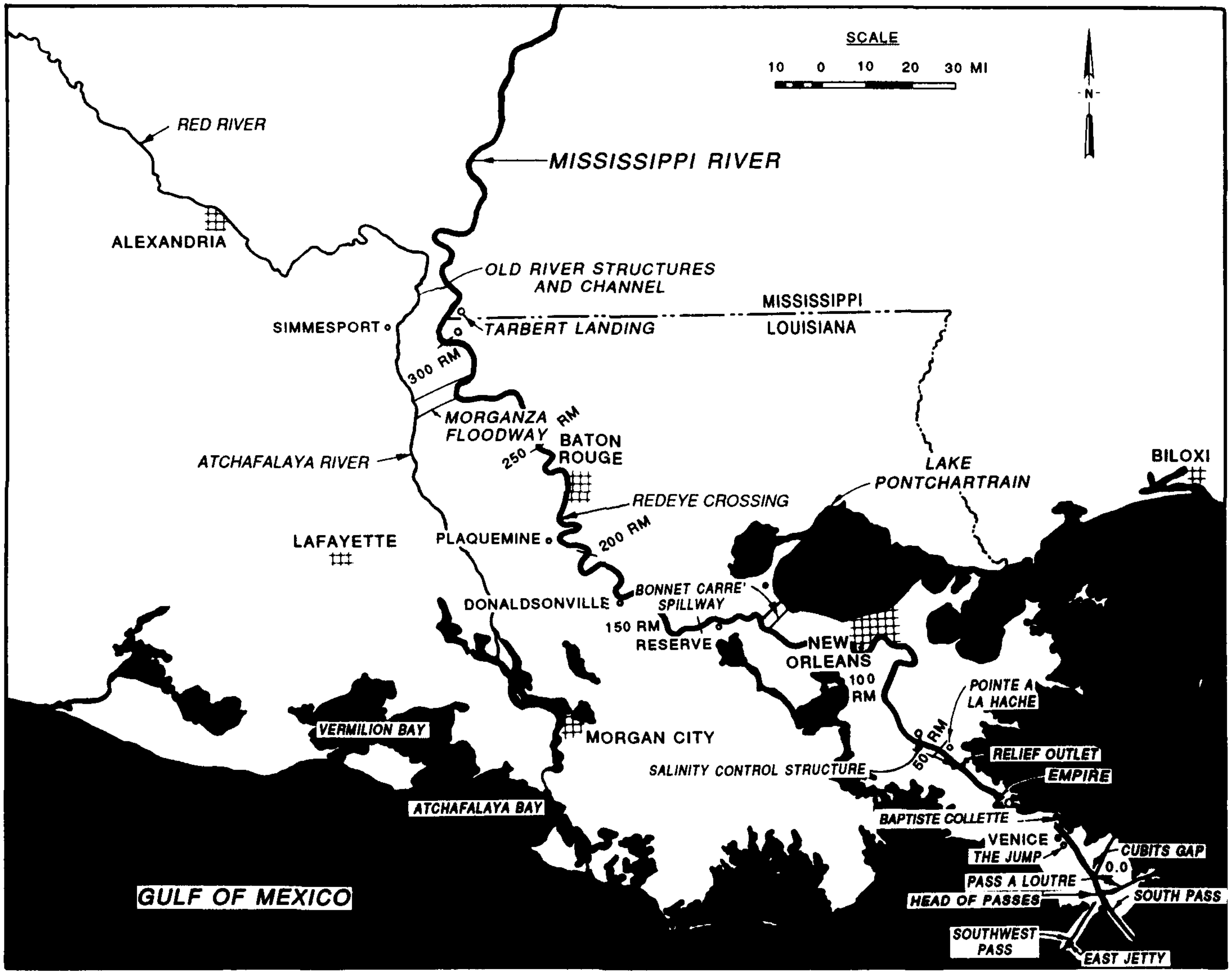

Figure. 1. Location map

LOWER MISSISSIPPI RIVER

TARBERT LANDING TO EAST JETTY SEDIMENTATION STUDY

Numerical Model Investigation

PART I: INTRODUCTION

The Prototype

1. About 330 miles* of the lower Mississippi River between Tarbert Landing and East Jetty, at the mouth of Southwest Pass, were included in this study (Figure 1). Flow into this reach of the lower Mississippi River is regulated by the Old River Control Structures, which became operational in 1963. These structures control the distribution of water and sediment into the lower Mississippi and Atchafalaya Rivers, allowing 30 percent of the combined flow from the Mississippi and Red Rivers to flow into the Atchafalaya. Prior to construction of these structures, the percentage of flow from the Mississippi into the Atchafalaya had been steadily increasing.

2. Tarbert Landing is located just downstream from the Old River Control Structures at river mile 306. It is the primary water and sediment discharge monitoring station for this reach of the Mississippi River. The US Army Engineer District, New Orleans, computes daily sediment and water discharge at this station.

3. Within the reach from Tarbert Landing to Baptiste Collete (river mile 11.4) are two major and one minor flood-control diversion structures. The Morganza Floodway, at river mile 280, diverts flow to the Atchafalaya River. The Bonnet Carre Spillway, at river mile 128, diverts flow into Lake Pontchartrain. These structures are operated as necessary to ensure that the discharge at New Orleans does not exceed 1,250,000 cfs. The Bohemia Spillway, located between river miles 44 and 33, is a natural levee that acts as a spillway during very high discharges, but carries only a small percentage of the flow. Aside from these diversions, the Mississippi River is contained by

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 5.

levees or high bluffs until it reaches Baptiste Collette, a natural distributary.

4. Downstream from Baptiste Collette are several additional natural distributaries. The major distributaries—Pass a Loutre, South Pass, and Southwest Pass—all start at Head of Passes, at river mile 0.0. The primary navigation channel is Southwest Pass. East Jetty at river mile -19.6 is at the mouth of Southwest Pass.

5. The Mississippi, in its natural state, was a heavy sediment-bearing river. Sediment loads increased in the early nineteenth century when the river's drainage basin was changed from predominately grasslands and forests to intensively developed agricultural lands. A series of river control structures constructed since 1950 has reversed this trend, and sediment yield has been on the decline. Reservoirs, which serve as sediment retention structures, were constructed between 1953 and 1967 in the Kansas and upper Missouri river basins. During these same years channel stabilization works were constructed on the lower Missouri River reducing sediment supply from bank erosion. Reservoirs and bank stabilization works constructed on the Arkansas River between 1963 and 1970 further reduced sediment supply. In addition, improved land-use practices and the placement of numerous streambank protection structures throughout the Mississippi River basin have reduced sediment loads. A more detailed history of sediment supply can be found in Keown, Dardeau, and Causey (1981).

Purpose of the Model Study

6. The numerical model study reported herein was conducted to address questions relating to sediment problems on the lower Mississippi River. The model was originally developed in 1984 to answer questions related to the sediment transport characteristics associated with a proposed sill to reduce salt-water intrusion at low river flows. The sill was to be constructed from dredged material and extend across the entire river width. The model was used to determine the rate of sill erosion at high flows, and the effect of the sill on water-surface elevations. During the 1988 drought, a sediment sill was actually constructed and hydrographic surveys were conducted during washout of the sill between November 1988 and February 1989. These data were used to test the predictive capability of the numerical model. Another question

was related to an apparent degradation trend in the Mississippi River, in the vicinity of New Orleans, that was first observed after the 1973 flood. The New Orleans District wanted to know if this was a short-term effect of the 1973 flood or a long-term trend. The model was also used to address questions relating to the evaluation of sediment diversion schemes and their effect on dredging in Southwest Pass. The initial scheme tested was to divert water and sediment at Bonnet Carre on a year-round schedule. Another scheme tested was to divert 70 percent of the Mississippi River; three diversion locations between river miles 44 and 81 were tested. The model was also used to evaluate dredging and flow diversion strategies at Cubits Gap; results are described in another report (Copeland 1991). It was also used to evaluate the effects of proposed dike fields on dredging at Redeye Crossing (river mile 224).

PART II: THE MODEL

Description

7. The TABS-1 generalized computer program was used to develop the numerical model for this study. The program was initially developed by Mr. William A. Thomas at the US Army Engineer District, Little Rock, in 1967. Further development at the US Army Engineer Hydrologic Engineering Center by Mr. Thomas led to the widely used HEC-6 generalized computer program for calculating scour and deposition in rivers and reservoirs (USAEHEC 1991). Additional modification and enhancement to the basic program by Mr. Thomas at the US Army Engineer Waterways Experiment Station (WES) led to the TABS-1 program currently in use (Thomas 1980, 1982). The program produces a one-dimensional model that simulates the response of the riverbed profile to sediment inflow, bed material gradation, and hydraulic parameters. The model simulates a series of steady-state discharge events and their effect on the sediment transport capacity at cross sections and the resulting degradation or aggradation. A more detailed description of the program is found in Appendix A.

8. The numerical model of the lower Mississippi River was developed in stages. The first model was the Long-Term-Trend (LTT) model, which was used to evaluate long-term trends and the proposed sediment sill. The Southwest Pass (SWP) model was developed later to evaluate the effect of various diversion schemes on dredging in Southwest Pass. Finally, the models were combined into a single numerical model which incorporated more recent hydrographic survey and sediment data, using the most recent enhanced version of TABS-1. This model was used to calculate bed volume changes between 1975 and 1983 and washout of the sediment sill. The new combined model was also used to predict the effect of proposed dikes in Redeye Crossing.

9. The LTT model extended from Venice to Tarbert Landing. The initial model geometry was based on the 1961-63 hydrographic survey.* Roughness coefficients were adjusted such that the water-surface elevations calculated by the model matched average stages at several gages. The model included three sand size classes from very fine to medium sand. Suspended sediment

* Elevations from the 1963 hydrographic survey were adjusted to account for datum changes that occurred with the resurvey of gage datums in 1974 and 1975.

measurements at Tarbert Landing were used to determine sediment inflow. The initial bed material gradation was adjusted so that the model simulated measured bed volume changes between 1963 and 1975.

10. The SWP model extended from East Jetty to Reserve (river mile 140.8). The initial geometry for this model was based on the 1973-75 hydrographic survey. Roughness coefficients were adjusted such that calculated water-surface elevations matched average stages at several gages. Flow distributions into the various distributaries were based on data developed by the New Orleans District and WES and reflect Supplement II improvements (USACE-NOD 1984) for hydrographs used to predict future conditions. The model included clay, four silt size classes, and three sand size classes from very fine to medium sand. Sand inflow was determined using the results of the LTT model. Clay and silt inflow was determined from measurements at Tarbert Landing, Carrollton (river mile 102.7), and Belle Chasse (river mile 76.0). The calculated bed material gradation for 1975 from the LTT numerical model was used as the initial bed material gradation for most of the SWP model. An iterative process was used to develop an initial gradation for the portion of the study downstream from Venice. Sediment deposition coefficients for silt and clay were adjusted until calculated dredging quantities for an eight-year period (1975-82) corresponded to reported values.

11. The new combined model extended from East Jetty to Tarbert Landing. The initial geometry was based on the 1973-75 hydrographic survey with an optional geometry based on the 1983-85 hydrographic survey. The 1983-85 hydrographic survey did not include elevations above the waterline; these cross sections were extended using the 1973-75 hydrographic survey. Roughness coefficients were adjusted to 1983 stage-discharge curves and were found to be essentially the same as those used in the previous models. Flow distribution into the various distributaries was based on data developed by the New Orleans District and WES, and reflects Supplement II improvements for historical hydrographs after 1988 and for hydrographs used to predict future conditions. The model included eight sediment sizes ranging from clay to medium sand. Suspended sediment measurements at Tarbert Landing were used to determine sediment inflow. The initial bed material gradations were based on 1975 or 1984 calculated results. Sediment deposition coefficients for silt and clay came from the SWP model.

Channel Geometry

12. The new combined numerical model had 97 cross sections, with distances between sections ranging from 0.5 to 8 miles. At early stages of model development, during long-term simulations, an occasional cross section would show significant deposition. This was always at a bend and was attributed to an enlarged cross-sectional area created by two-dimensional flow patterns. Such cross sections were considered unrepresentative of one-dimensional assumptions inherent in the model and were removed.

13. The numerical model dredges Southwest Pass and Cubits Gap each year on the recession of the annual hydrograph when the discharge falls below 700,000 cfs. The model dredges the navigation channel at each cross section to templates similar to sections from the 1983-85 hydrographic survey. In general, the navigation channel is 750 ft wide and 45 ft deep. Dredged material is removed from the model. Other dredging operations, including returning dredged material to the water column, may be incorporated into the numerical model to study specific problems. This option was incorporated to study proposed dikes at Redeye Crossing.

Hydrographs

14. Discharge hydrographs are simulated in the numerical model by a series of steady-state events. The duration of each event is chosen such that changes in bed elevation, due to deposition or scour, do not significantly change the hydraulic parameters during that event. At relatively high discharges, durations need to be short; time intervals as low as one day were used in this study. At low discharges, the time intervals may be extended; time intervals up to 31 days were used in this study.

15. A hydrograph simulated by a series of steady-state events of varying durations is called a histograph. The histographs used in this study were based on mean daily flow measurements at Tarbert Landing. The years 1963-75 were used to adjust the numerical model, and the years 1975-89 were used for model testing. Discharges in the Mississippi River were reduced to account for measured flows diverted through the Morganza Floodway and the Bonnet Carre Spillway. These structures were operated to maintain a maximum flow of 1,250,000 cfs at New Orleans. The Morganza Floodway was operated in 1973 and

the Bonnet Carre Spillway was operated in 1973, 1975, 1979, and 1983.

16. The sediment diversion schemes were evaluated using the 1975-82 histograph as a representative flow period. This period encompassed a range of annual flow events including a major flood. The flood duration curve for this period is compared with the 54-year period of record flow duration curve in Plate 1. A 32-year period was simulated in the numerical model by running the 1975-82 histograph four times.

Distributary Flow Distribution

17. Downstream from Pointe a la Hache (river mile 49) distributaries begin to occur. The Bohemia Spillway, located between river miles 33 and 44, allows some flow to leave the river when the water-surface elevation exceeds 7 ft. At a maximum discharge of 1,250,000 cfs, it is estimated that the outflow would be about 30,000 cfs or 2 percent of the total. The effect of the outlet was considered insignificant and was therefore not simulated in the study. Distributaries simulated in the model include: Baptiste Collette (river mile 11.4), the Jump (river mile 10.5), Cubits Gap (river mile 3.0), Pass a Loutre and South Pass (river mile 0.0), and outlets along Southwest Pass. In addition, flow is lost over the natural levees downstream from Venice (river mile 10.7) during high flows.

18. Monthly distributary flow distributions prior to Supplement II improvement between Head of Passes and Baptiste Collette were determined by the New Orleans District (USACE-NOD 1984). These data were used with average monthly discharges and extrapolated to develop a flow percentage rating curve for each distributary between miles 0.0 and 11.4. In the numerical model, Cubits Gap and flows over the natural levees between Venice and Head of Passes were combined, and Pass a Loutre and South Pass flows were combined.

19. Distributary flow distribution in Southwest Pass prior to Supplement II improvements was determined from four sets of flow measurements taken between 1964 and 1976. Measurements were taken at Head of Passes and at the major outlets, but not at East Jetty. Conditions at specific outlets changed during the period of record. Flow through several outlets increased significantly between 1964 and 1970. The following criteria were used to establish outlet flow percentages for the numerical model:

a. If the 1975 hydrographic survey showed a dike across an outlet and no measurements were taken in 1976 then the outlet was considered closed.

b. If a significant increase in outflow occurred between 1964 and 1970 and then the flow percentage stabilized over the next six years, the last three measurements were averaged to determine the outflow.

c. If there was no consistent trend in the measured data, all available measurements were averaged.

Based on the measured data it was determined that about 8.5 percent of the total flow leaves the channel through outlets in Southwest Pass. It was estimated that an additional 1.5 percent of the flow passes over the south natural levee between river miles -4.8 and -9.0. In the numerical model, flows from outlets and overbanks in Southwest Pass were combined and discharged at river miles -4.5 and -9.8. The rating table (flow distribution percentage) prior to Supplement II improvements for the distributaries simulated in the numerical model is given in Table 1.

Table 1

Distributary Flow Distribution Percentage

Prior to Supplement II Improvements

Distributary	Location River Mile	Discharge, 1000 cfs, Upstream from River Mile 12.4 200	500	800	1250
Baptiste Collette	11.4	4	4	4	4
The Jump	10.5	5	5	5	5
Cubits Gap and Overbank Flows	3.0	16	16	22	27
South Pass and Pass a Loutre	0.0	45	45	41	38
Joseph Bayou and Overbank Flows	-4.5	3	3	3	3
Outlets 11.8W W-1 W-2	-9.8	7	7	7	7
East Jetty	-19.6	20	20	18	16

20. Flow distributions downstream from Venice for historical hydrographs after 1988 were adjusted to reflect Supplement II improvements. Distributary flow distributions for the 32-year hydrograph, used to evaluate proposed diversion schemes, also reflect Supplement II improvements. These

improvements include closing some outlets, improving dike fields, and raising natural levees. Supplement II flow distributions (Table 2) were obtained from a TABS-2 two-dimensional numerical model study conducted at WES (Richards and Trawle 1988).

Table 2

Distributary Flow Distribution Percentage With Supplement II

Distributary	Location River Mile	Discharge, 1000 cfs, Upstream from River Mile 12.4			
		200	640	900	1300
Baptiste Collette	11.4	6	6	6	5
The Jump	10.5	9	9	8	6
Cubits Gap and Overbank Flows	3.0	9	9	14	21
South Pass and Pass a Loutre	0.0	50	50	46	42
Joseph Bayou and Overbank Flows	-4.5	2	2	2	2
Outlets 11.8W W-1 W-2	-9.8	2	2	4	7
East Jetty	-19.6	22	22	20	17

Water Temperature

21. Water temperature was obtained from USGS Water Quality Records (US Geological Survey 1975-1983). The seven water quality gages used were: Venice, Belle Chasse, Carrollton, Luling Ferry (river mile 120.6), Union (river mile 168.0), Plaquemine (river mile 208.0), and St. Francisville (river mile 266.0). Available monthly values at each gage were averaged to obtain a representative value for the entire study reach. Sufficient data were available to determine a temperature for every month between 1967 and 1981. Average values over the entire period of record were used for other years. Average water temperatures ranged from 42°F in winter to 84°F in summer.

Downstream Water-Surface Elevations

22. An analysis of stages at East Jetty and routed discharges from

Tarbert Landing indicated a poor stage–discharge correlation. Therefore downstream water-surface elevations for the model were based on average monthly stages. These are shown below (USACE-NOD 1984).

	Stage, ft, NGVD		Stage, ft, NGVD
January	0.6	July	2.0
February	0.7	August	1.9
March	1.5	September	1.9
April	2.0	October	1.3
May	2.1	November	1.0
June	1.9	December	1.0

Channel Roughness

23. Channel roughness in the lower Mississippi River varies with stage and discharge. Gage records indicate that roughness is different on the rising limb of the hydrograph from that on the falling limb. This is apparent from the hysteresis in stage–discharge rating curves, which is often greater than can be accounted for by unsteady flow alone (Huval 1979). Measured stage–discharge curves at Tarbert Landing for 1963 and 1973 are shown as an example in Plate 2. The numerical model simulates average roughness and therefore does not account for these annual roughness differences. Manning's roughness coefficients in the numerical model were adjusted to six gages between Venice and Tarbert Landing. In the LTT and SWP models, 1962–73 average stages were used to determine Manning's roughness coefficients. Adopted values of Manning's roughness coefficients varied between 0.016 and 0.026. Calculated water-surface elevations compare favorably with measured stages as shown in Figure 2. In the new combined model, 1983 stage–discharge curves were used. Adjustments to the Manning's roughness coefficients in the numerical model to match average recorded stages for a range of discharges were minor and there was no observed trend with respect to the old model. Adopted values of Manning's n varied between 0.014 and 0.026. Calculated water-surface profiles for a range of discharges with the adopted roughness coefficients are compared with average 1983 stages at various gages in Figure 3.

24. Specific gage curves for the years between 1975 and 1989 were developed at Carrollton, Baton Rouge (river mile 228), and Tarbert Landing (Plates 3–5). These curves were developed to identify any long-term

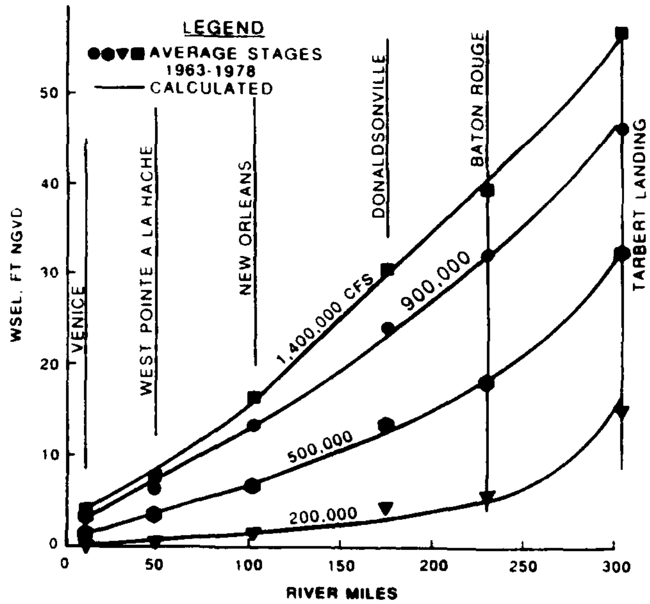

Figure 2. Measured and calculated stages, 1962-73

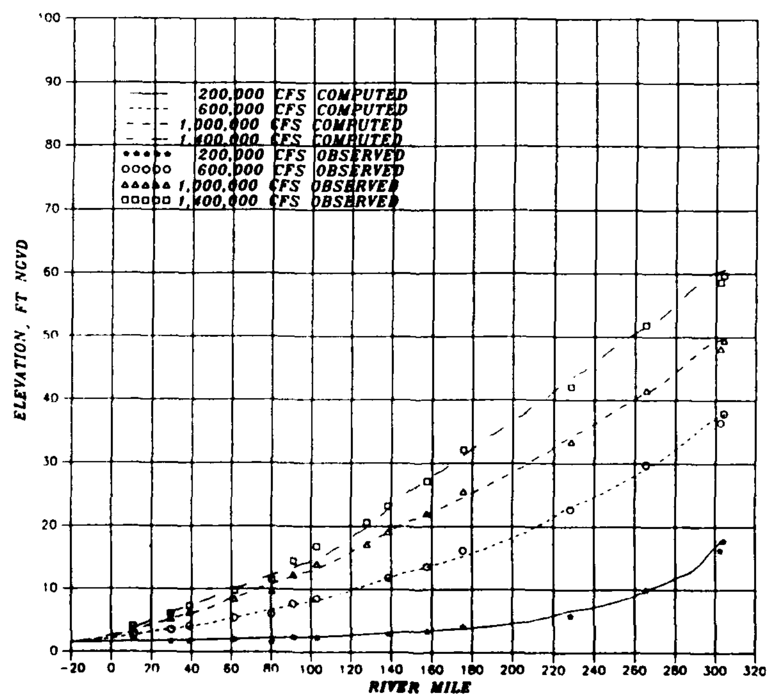

Figure 3. Measured and calculated stages, 1983

aggradation or degradation trends. The discharge interval was 50,000 cfs. Regression lines through the data at all three gages indicate a slight decrease in stage-discharge relationships for all discharges. It should be noted that the decrease is contained within the data scatter, which indicates that other factors such as the hysteresis in the annual hydrograph are more significant in terms of variation in stage-discharge relationships.

Sediment Inflow at Tarbert Landing

25. Suspended sediment concentration in the Mississippi River at Tarbert Landing has varied considerably over the last 40 years. Average annual suspended sand concentrations from 1950 to 1990 are shown in Figure 4. The

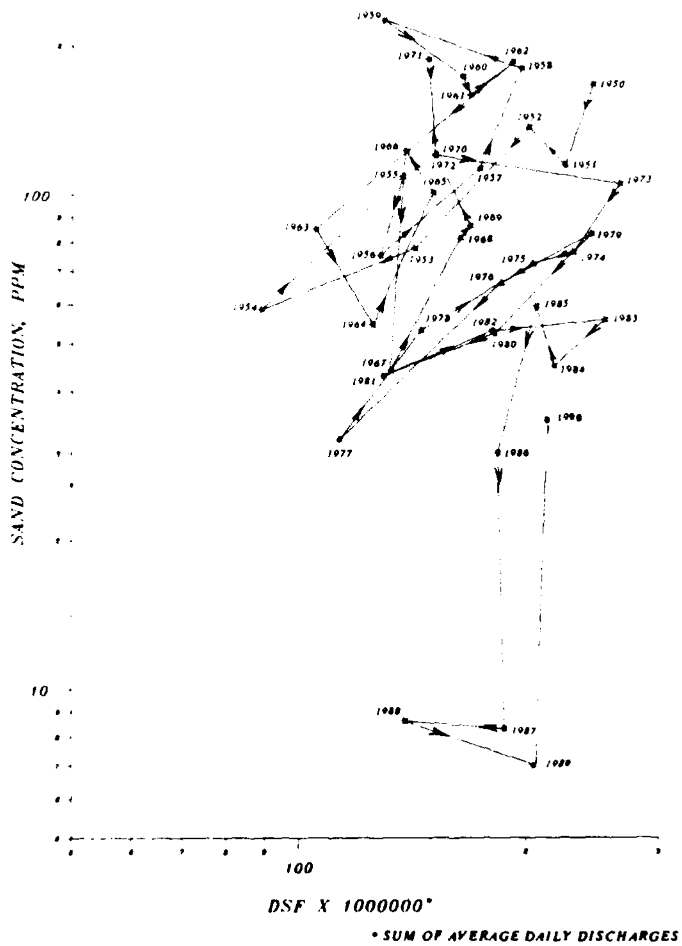

Figure 4. Average annual measured sand concentration at Tarbert Landing, 1950–90

five years prior to the 1961-63 hydrographic survey had very high sediment concentrations. In 1973 and thereafter, sediment concentrations were lower, displaying a relatively steady relationship with discharge until 1983. A slight decrease in measured suspended sand load occurred in 1983, followed by a significant decrease in measured suspended sand load in 1986.

26. The general decline in suspended sediment concentration in the lower Mississippi River since 1950 has been attributed to construction of a series of reservoirs and bank stabilization works on the Mississippi River and its tributaries (Keown, Dardeau, and Causey 1981). The cause of the decline in 1986 is uncertain, but several possible explanations have been advanced. One possibility is that the total sand concentration, calculated from samples, may be influenced by a 1983 change in the sampling procedure at Tarbert Landing. In 1983 the total number of point samples was reduced from 40 to 8. This included changing the maximum sample depth to 70 percent depth, instead of 90 percent depth. In April 1990, the number of point samples was increased to 12, including a sample at 90 percent depth. Measured sand in 1990 increased significantly. In 1990, tests were conducted alternating 40 and 12 point samples and there did not appear to be any difference between the two sets of data. A recent investigation of sampling techniques by the New Orleans District revealed no irregularities. A second explanation for the decline in sand concentrations is related to new structures operating at Old River (river mile 312). The Auxiliary Structure at Old River was opened in December 1986. This structure was designed to divert a greater portion of the sand load than the Old River Control Structures. Opening of the Old River Hydropower structure in 1990 is also expected to have an effect on sand load downstream at Tarbert Landing. It has also been suggested that a greater percentage of flows in the lower Mississippi have been coming from the Ohio River since 1985. The Ohio River basin has a much lower sediment concentration than the Missouri River basin, which has had lower than normal flows since 1985. Other factors that affect sediment concentrations include the rate of rise of the annual hydrograph and the temperature of the water.

27. Suspended sand data from Memphis, Vicksburg, and Natchez were evaluated to determine if the decline in sand concentration was also occurring at these upstream gaging stations. Sampled suspended sand data at Memphis between 1976 and 1988 were obtained from USGS Water Supply Papers and are plotted in Plate 6. Any possible downward trend in suspended sand load is

obscured by the data scatter. An analysis of suspended sand load at Vicksburg and Natchez was conducted by the New Orleans District. Preliminary results indicate a possible downward trend at both Natchez and Vicksburg, similar to the trend at Tarbert Landing with a year or so lag.

28. Sediment inflow curves were developed from data at Tarbert Landing for each size class. Size class data were available for the years 1965-89. Sediment inflow curves for 1959-64 were based on size class percentages from 1971, which was a year with similarly high sediment concentrations. Measured data from 1965 to 1972 were combined to develop a single sediment rating curve for 1963-72. Distinctive curves were developed for 1973-74, 1975, 1976-82, 1983-85, and 1986-89. Sediment discharge rating curves for certain years were distinctive even though the average annual concentration, as shown in Figure 4, may have indicated similarity.

29. The unmeasured load has been estimated at 15 percent of the measured load. This value was first suggested by Dr. Hans Einstein in the 1960's. Based on their subsequent studies and investigations, the New Orleans District has concluded that the 15 percent estimate is appropriate (USAED New Orleans 1980). Total sand loads were determined for the model by increasing the suspended measurements of each sand size class by 15 percent.

30. Considerable scatter occurs in the total sand load data as shown in a representative set of sediment inflow curves (Plates 7-9). An optical fit was used to establish the rating curve. The sediment inflow values used in the model are given in Table 3.

Silt and Clay Inflows

31. Suspended sediment measurements at Tarbert Landing, Carrollton, and Belle Chasse were used to develop a silt and clay inflow rating table for the numerical model. Tarbert Landing measurements taken between October 1972 and August 1976 were used. During this period measurements were made weekly except at high flows when measurements were taken more frequently. The Carrollton measurements were taken at two- to three-day intervals during the peak and recession of the 1973 and 1975 flood hydrographs and during the low flow months of 1976. The Belle Chasse measurements were taken monthly between October 1979 and August 1982.

32. At high flows there appeared to be little correlation between water

Table 3

Total Sand Inflow at Tarbert Landing

Date	Water Discharge 1,000 cfs	Sediment Discharge tons/day Very Fine Sand	Fine Sand	Medium Sand
1959-63	100	1,900	3,500	750
	300	26,400	33,800	7,810
	500	93,000	97,300	23,600
	800	296,000	257,000	65,100
	1,500	1,330,000	1,330,000	255,000
1963-72	100	400	100	30
	250	20,000	30,000	5,000
	500	70,000	63,000	13,000
	800	160,000	111,000	23,000
	1,500	500,000	200,000	50,000
1973-74	100	200	100	30
	200	2,000	1,400	300
	500	45,000	35,000	7,000
	800	75,000	80,000	20,000
	1,500	90,000	120,000	40,000
1975	100	100	100	30
	200	1,400	1,400	180
	500	40,000	35,000	2,000
	800	100,000	70,000	7,000
	1,500	120,000	100,000	20,000
1976-82	100	100	100	30
	200	1,400	1,400	230
	500	40,000	35,000	5,000
	800	80,000	70,000	13,000
	1,500	120,000	300,000	50,000
1983-85	100	115	115	35
	200	1,600	1,600	320
	500	35,000	35,000	4,400
	800	58,000	83,000	8,000
	1,500	104,000	138,000	20,000
1986-89	100	115	115	35
	200	1,000	800	320
	500	8,000	3,500	1,400
	800	22,000	8,000	2,800
	1,500	86,000	23,000	7,500

and fine sediment discharges. At Tarbert Landing fine sediment concentrations were significantly higher on the rising limb of hydrographs than during peak or recession flows. Concurrent measurements taken at Carrollton and Belle Chasse indicated an increase in fine sediment concentration in a downstream direction at high flows. This trend was reversed at low flows when fine sediment concentrations were higher upstream. Three measurements at Belle Chasse and one at Tarbert Landing indicated periods of abnormal influxes of fine material. The Belle Chasse data were for three consecutive months. Possible sources for this material are eroding banks or upstream dredging operations. Tributary inflow is negligible. Except for these abnormal measurements, the poor correlation of fine sediment concentration and water discharge at high flows is attributed to the hysteresis effect associated with the rising and falling of the annual hydrograph. A curve through the measured data, determined optically, was deemed appropriate to obtain an average inflow for fine sediments (Plate 10).

33. The size class distribution of the fine load was developed from the Carrollton data. The 1973 and 1975 data were averaged to give a high flow distribution, and the 1976 data were averaged for a low flow distribution. The results are shown in Plate 11. The size class distribution was applied to the total fine load curve to obtain the fine load sediment rating table, Table 4.

Table 4

Fine Sediment Inflow

Water Discharge cfs × 10^3	Sediment Inflow tons/day Clay	Very Fine Silt	Fine Silt	Medium Silt	Coarse Silt
100	8,800	400	400	400	313
300	93,600	7,800	7,800	10,400	10,400
500	272,200	25,800	30,100	47,300	51,600
600	316,200	30,600	35,700	61,200	66,300
1,500	624,000	104,000	117,000	195,000	260,000

Bed Material Gradations

34. There have been several sampling programs conducted in an attempt to

define the longitudinal variation of bed material in the lower Mississippi River between Tarbert Landing and East Jetty. In 1932, 68 bed samples were collected from the thalweg of the river between Tarbert Landing and Head of Passes (WES 1935). In 1975, 82 samples were collected over this same reach by the New Orleans District (Keown, Dardeau, and Causey 1981). Another sampling program was conducted by the New Orleans District in 1984 when 33 samples were collected between Tarbert Landing and Head of Passes. Nordin and Queen (1991) collected 118 bed material samples from the river thalweg in this reach in 1989. These data all indicated large variations in bed material gradation at various sections along the river. In an attempt to define the longitudinal variation of bed material, reach averages were calculated. These data are not directly applicable for use in the numerical model, which requires laterally averaged bed material gradations and which treats erosion and deposition of sands and cohesive sediment differently.

35. Several bed material sampling programs have been conducted at specific gaging stations. These data are more applicable to the numerical model because laterally averaged gradations are obtained. Good data are available at Tarbert Landing where 283 bed material samples were collected between 1971 and 1977. Fourteen samples were collected at Carrollton in 1976 during a low flow period. Samples of the bed and dredged material areas near Head of Passes and in Southwest Pass have been collected at various times between 1968 and 1981. Demas and Curwick (1987) of the USGS collected bed samples at eight stations along the river between 1982 and 1985. They took bed samples monthly, at a minimum of three points laterally across the station. These data indicated significant lateral variation in bed material gradation at both crossings and bends (Plates 12 and 13). There appeared to be no significant variation of bed material gradation with discharge. Averages of these data account for lateral and temporal variations and are more representative for use in one-dimensional numerical models.

36. There were insufficient data to define the bed material gradation for the numerical model at the beginning of the historical simulation in 1963. Therefore, bed material gradation was used as an adjustment parameter in the model. The initial model gradation in 1963 was adjusted until measured and calculated 1963-75 cumulative bed volume changes between Venice and Tarbert Landing were similar. Simulations beginning in 1975 and 1983 were assigned initial bed material gradations in the model, based on calculated gradations

from the previous simulation. Sampled bed material gradations were used to verify gradations calculated using the numerical model.

PART III: MODEL ADJUSTMENT AND CIRCUMSTANTIATION

37. The primary adjustment parameters for the numerical model were: (a) the initial 1963 bed material gradation, (b) transport function, (c) erosion and deposition coefficients for silts and clays, and (d) sediment concentrations leaving the river via the distributaries. The initial bed gradation in the numerical model was adjusted until calculated and measured cumulative bed volume differences between 1963 and 1975 were similar. The sand transport function used in the numerical model was chosen based on its ability to reproduce measured transport rates at gages downstream from Tarbert Landing. Distributary sediment concentrations and cohesive erosion and deposition coefficients were adjusted to simulate the 1974-82 total dredging volumes in Southwest Pass.

Bed Volume Changes 1963-75

38. Prototype aggradation and degradation were determined by calculating the change in end areas using cross sections from the 1962-63 and 1973-75 hydrographic surveys. Cumulative bed volume differences, starting at Venice and continuing to Tarbert Landing, were determined (Figure 5). About 120 million cubic yards of degradation occurred in the first 120 miles between Venice and New Orleans. For the next 60 miles the river was essentially stable. About 90 million cubic yards of aggradation occurred in the last 120 miles. A net degradation of 30 million cubic yards occurred between Tarbert Landing and Venice.

39. An initial bed material gradation was developed from the measured data at Tarbert Landing and the 1975 bed material sampling program between Tarbert Landing and Head of Passes. The numerical model was used to calculate cumulative bed volume changes using known sediment inflow, water discharge, and initial geometry. Initial calculated results, shown in Figure 5, are significantly different from prototype measurements. It was apparent that the actual initial bed material gradation, which represents 1963 conditions, was significantly different than determined using the 1975 sampling data.

40. Generally, the bed gradation is dependent on sediment inflow and hydraulic parameters. However, it was determined that hydraulic parameters did not change significantly with bed gradation or even deposition and

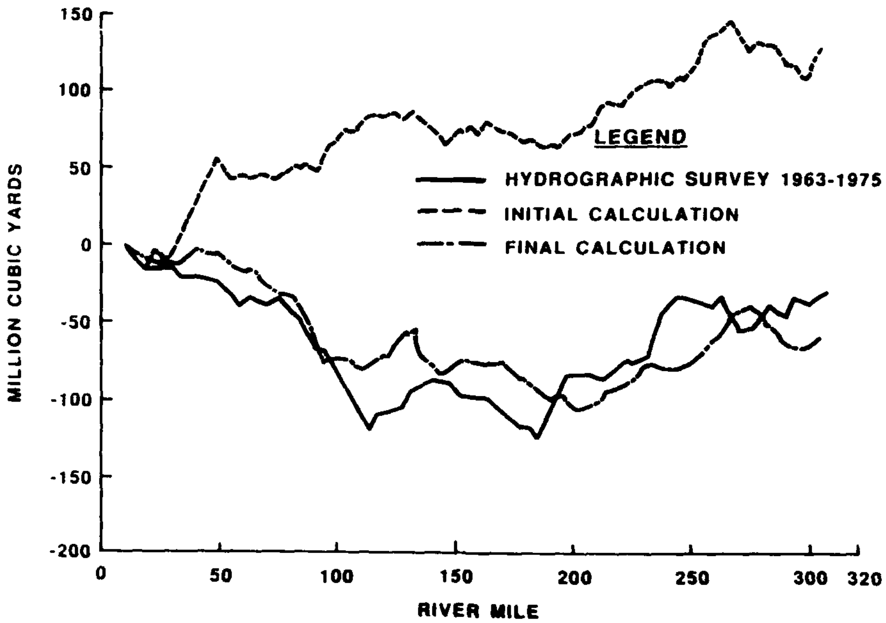

Figure 5. Measured and calculated cumulative bed volume changes, 1963-75

aggradation in the lower Mississippi River. Therefore, changes in bed gradation were primarily a function of changes in sediment inflow. The five-year period preceding 1963 was characterized by high sediment concentrations (Figure 4). The year 1971 had similarly high sediment concentrations. Since suspended sediment size class distribution data were not available for 1959-63, the 1971 sediment rating curves for each size class were used with the 1959-63 hydrograph to determine, iteratively, a bed gradation in equilibrium with the inflowing load. Initially, the bed gradation was based on the 1976 New Orleans and 1971-77 Tarbert Landing measurements. The five-year hydrograph was run and a new bed gradation determined. This process was repeated until the starting and ending bed gradations differed by less than 5 percent. The bed material gradation determined by the iterative procedure was used as the initial bed material gradation in the 1963-75 numerical model simulation. It was determined that this gradation still would not reproduce measured volume changes and, therefore, further adjustment was required.

41. The initial (1963) bed material gradation was adjusted in the numerical model until calculated and measured cumulative volume changes were similar. This was a tedious trial-and-error procedure; the comparison between final calculated and measured cumulative volume changes is shown in Figure 5.

Initial bed material gradations based on 1975 samples are compared with the adjusted bed material gradations in Figure 6.

Bed Volume Changes 1975-84

42. The numerical model simulation was extended to 1984, and calculated bed volume changes were compared with calculations of bed volume change using the 1973-75 and 1983-85 hydrographic surveys. Cross sections at the same river mile were used from the two hydrographic surveys to determine the change in cross-sectional area. A representative length was assigned to each cross section to determine bed volume change. When the change in cross-sectional area was relatively large, additional cross sections were added to decrease the representative length. A total of 150 cross sections were used in the calculations. The comparison of calculated cumulative volumes using the hydrographic surveys and the numerical model is shown in Figure 7. Between 1975 and 1984 the hydrographic survey comparison indicated an accumulation of 11 million cubic yards (5.2 percent of the incoming sand load) in the 300-mile reach and the numerical model indicated an accumulation of 21 million cubic yards (10.0 percent of the incoming sand load). For the entire 300-mile reach, 21 million cubic yards of accumulation relates to an average annual increase in bed elevation of about 0.01 ft.

43. The calculated longitudinal distribution of bed volume changes was dissimilar. The numerical model indicated general aggradation between river miles 80 and 105. The hydrographic survey comparisons indicated no significant change between those river miles. On the other hand, the hydrographic survey data indicated a general aggradation trend between river miles 160 and 250, while the numerical model indicated a slight degradation trend. Degradation trends between river miles 10 and 60 and between 270 and 306 were indicated by both the hydrographic survey comparison and the numerical model.

44. A problem associated with comparing hydrographic surveys is that cross sections surveyed at a particular river mile may not have been surveyed at the same time of year, or at the same location on the annual hydrograph, and therefore may not be homologous. Differences in surveyed cross-sectional areas may be attributed to normal annual variations caused by the rising and falling of the annual hydrograph. For instance, the cross sections between river miles 80 and 105 were surveyed on the rising limb of the 1975

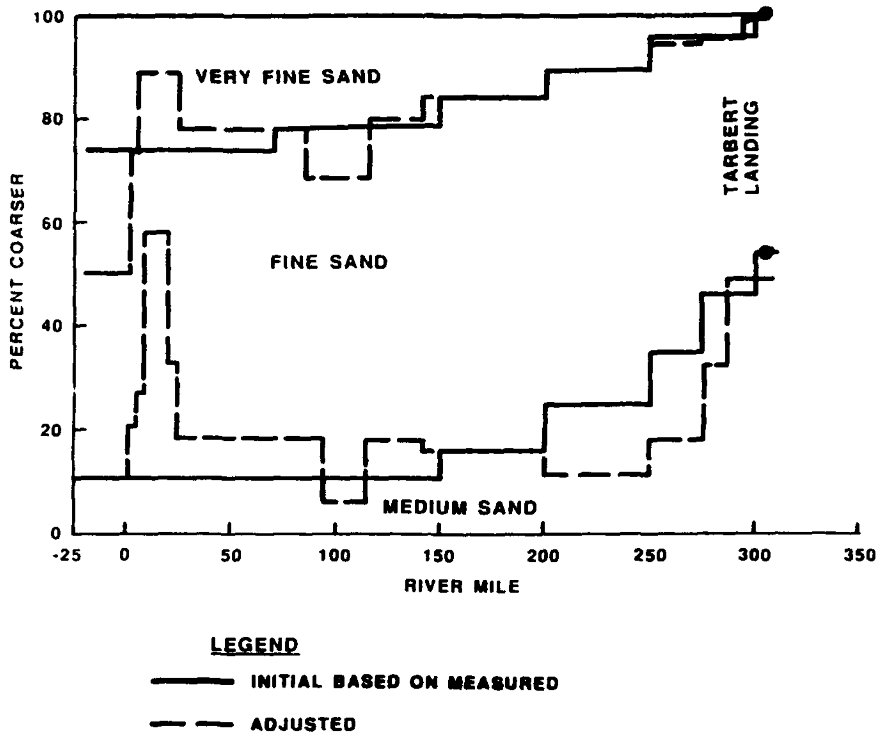

Figure 6. Bed material gradation, Mississippi River

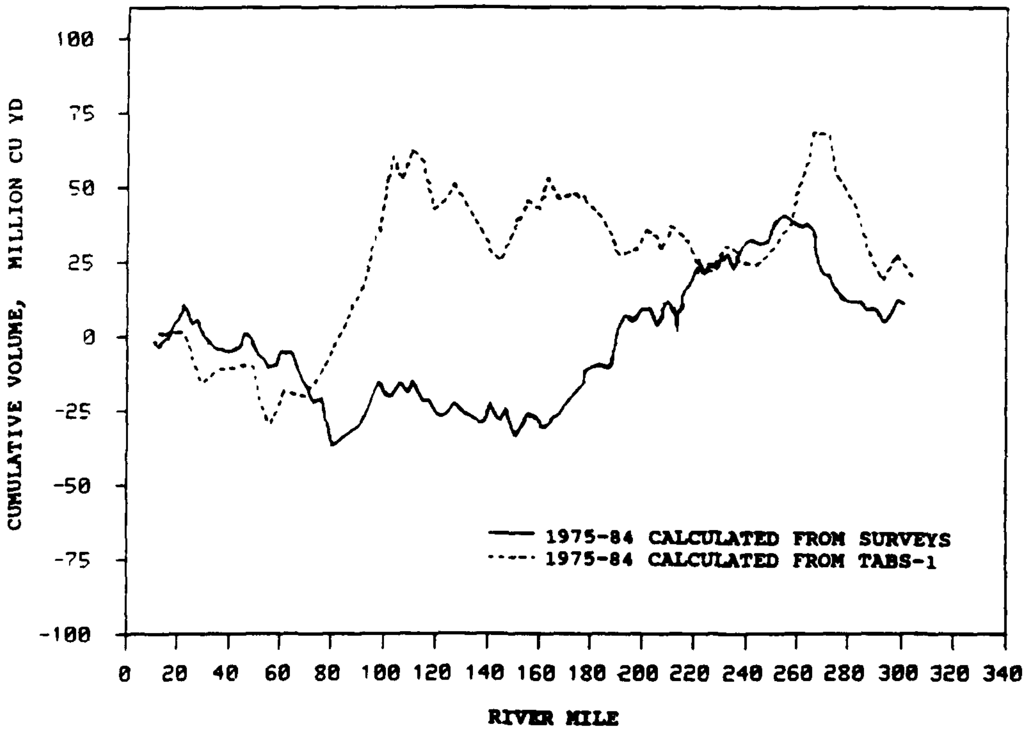

Figure 7. Surveyed and calculated cumulative bed volume changes, 1975-84

hydrograph, when the discharge was between about 600,000 and 850,000 cfs, and at the peak of the 1984 hydrograph, when the discharge was between 1,000,000 and 1,150,000 cfs. In addition, dredging at specific crossings in relation to the hydrographic survey may influence the comparison. For instance, at Redeye Crossing (river mile 224) the survey taken in February 1974 had bed elevations in the navigation channel that were about 20 ft higher than bed elevations from the survey taken in August 1983 after dredging.

45. Because of the differences in cumulative bed volume calculations, it was concluded that the hydrographic survey data could not be used to verify the performance of the numerical model at specific locations along the study reach. Annual bed elevation oscillations at cross sections appear to be more significant than long-term bed volume changes in the 300-mile study reach. However, the numerical model result indicating insignificant total bed volume change in the study reach between 1975 and 1984 is supported by the hydrographic survey data.

46. The numerical model was circumstantiated by comparing measured and calculated transport rates at Carrollton (Figure 8) and Belle Chasse (Figure 9). The Laursen transport function, modified by Madden (USAEHEC 1977), was used in this study because calculated and measured transport for each sand size class compared favorably. These comparisons were deemed adequate, although calculated transport at Carrollton was high during the 1973 and 1975 flood events. The Carrollton measurements were taken at peak and recession flows. Measurements at Tarbert Landing indicate that sediment concentration is significantly higher during the rising limb of the hydrograph.

Calculated Bed Material Gradations

47. The initial (1963) bed material gradations were used as an adjustment parameter for the numerical model. Calculated bed material gradations from the 1963-89 historical simulation were compared with sampled bed material gradations as a means of model circumstantiation. Comparisons were made for data collected in 1975 and 1984 by the New Orleans District, between 1982 and 1985 by the USGS (Demas and Curwick 1987), and in 1989 (Nordin and Queen 1991). Bed material samples collected by the New Orleans District in 1975 are compared with the calculated bed material gradations in Figure 10. Prototype samples that contained more than 10 percent fines were excluded because they

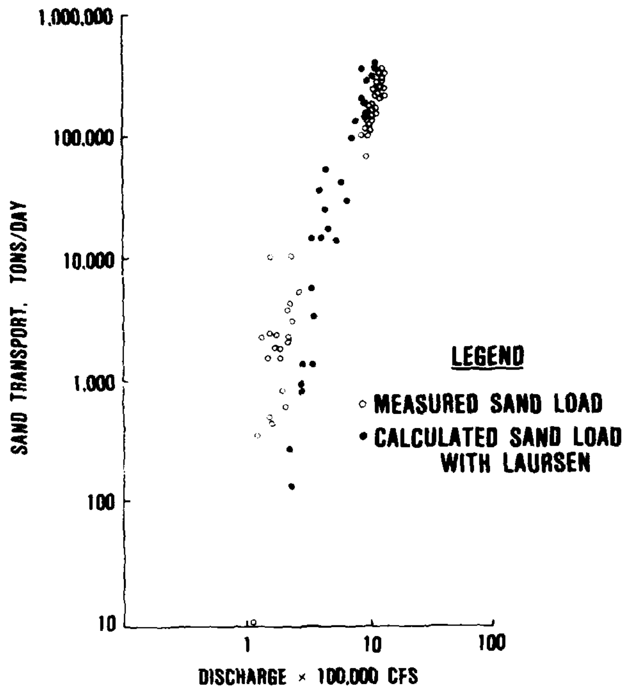

Figure 8. Measured and calculated sand discharge at Carrollton, river mile 102.7

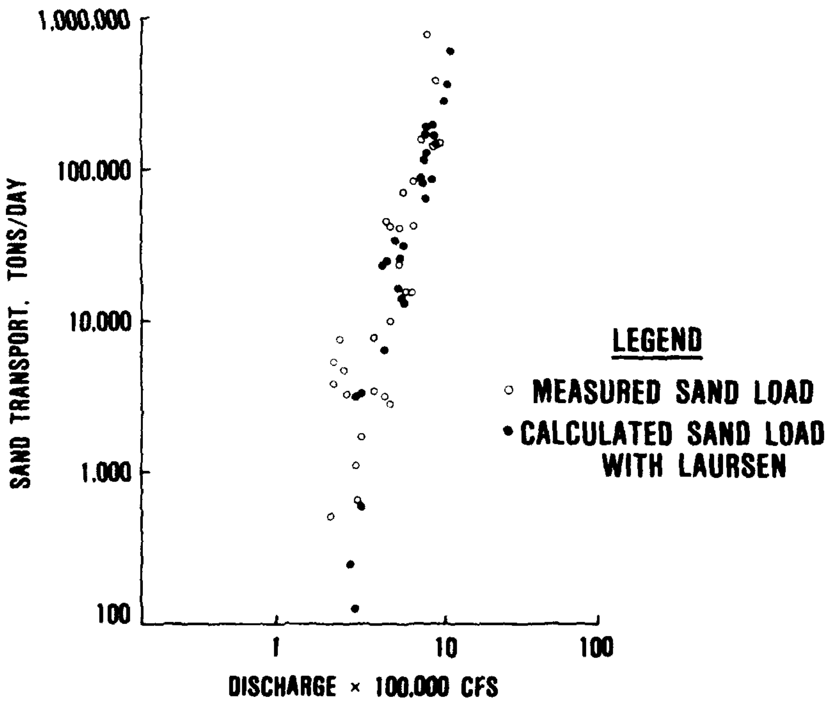

Figure 9. Measured and calculated sand discharge at Belle Chasse, river mile 76.0

are not representative of the bed with respect to calculations of exchange with the water column. This included most of the samples downstream from river mile 150. The numerical model is based on the assumption that 100 percent of the bed surface is available for exchange. This may create a simulation problem when the prototype bed is partially composed of gravel or erosion-resistant cohesives. The model compensates for this inconsistency by developing a coarser bed composition. This characteristic is noticeable in the model for cross sections between river miles 10 and 20 in Figures 10 and 11.

48. Samples collected by the New Orleans District in 1984 are compared with the calculated 1984 bed gradation in Figure 11. Very few of the 1984 samples contained significant quantities of fine material. Bed material gradations calculated for 1984 were used as initial conditions for the optional 1983-85 initial bed geometry file.

49. The USGS collected bed samples at eight stations along the river between 1982 and 1985. These data indicated a significant lateral variation in bed material gradation at both crossings and bends. There appeared to be no significant variation of bed material gradation with discharge. Averages of these data account for lateral and temporal variations and are more representative for use in one-dimensional numerical models. Average USGS gradations are compared with bed gradations calculated with the numerical model for a 1983 high flow in Figure 12.

50. In 1989, Nordin and Queen (1991) collected bed samples from the thalweg of the Mississippi River, between Cairo (river mile 956) and Head of Passes, for the Lower Mississippi Valley Division of the US Army Corps of Engineers (Figure 13). Samples were collected from the thalweg of the river in order to compare 1989 bed gradations with thalweg samples collected in 1932. The samples were collected using the same type of bed sampling equipment as that used in 1932. Between Tarbert Landing and Venice, Queen and Nordin found little change in median grain size between 1932 and 1989 samples; however, they found the bed to be more uniform with less very fine sand.

51. Considering the variability of the bed in the Mississippi River, calculated average bed material gradations from the numerical model are reasonable. The variability in the bed material data sets is attributed to localized response to turbulent fluctuations and to hydraulic conditions which

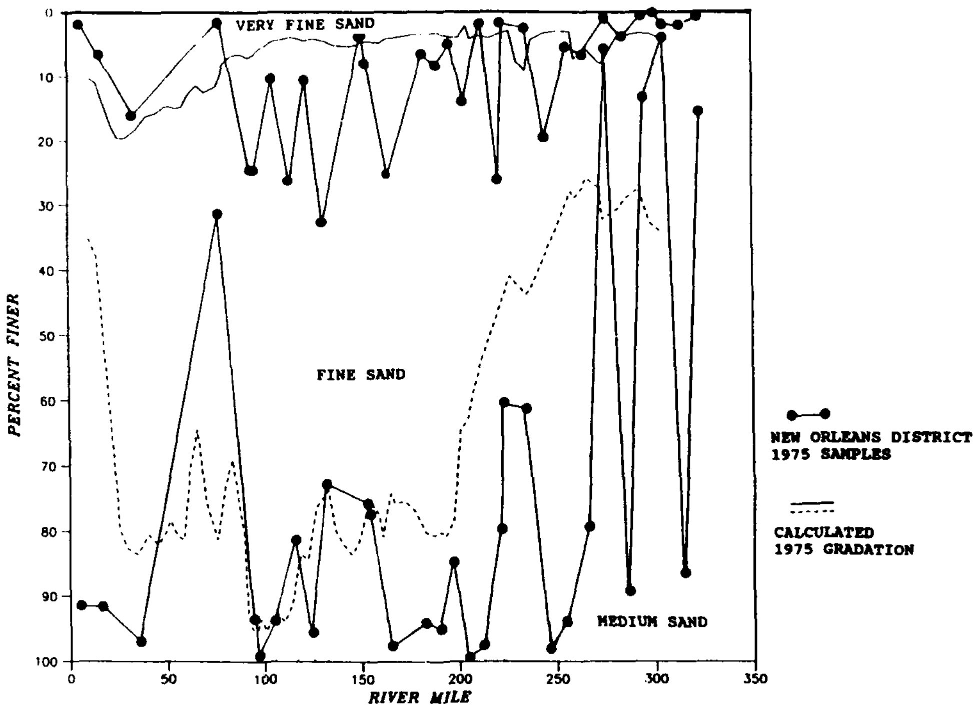

Figure 10. Comparison of 1975 calculated and sampled bed gradations

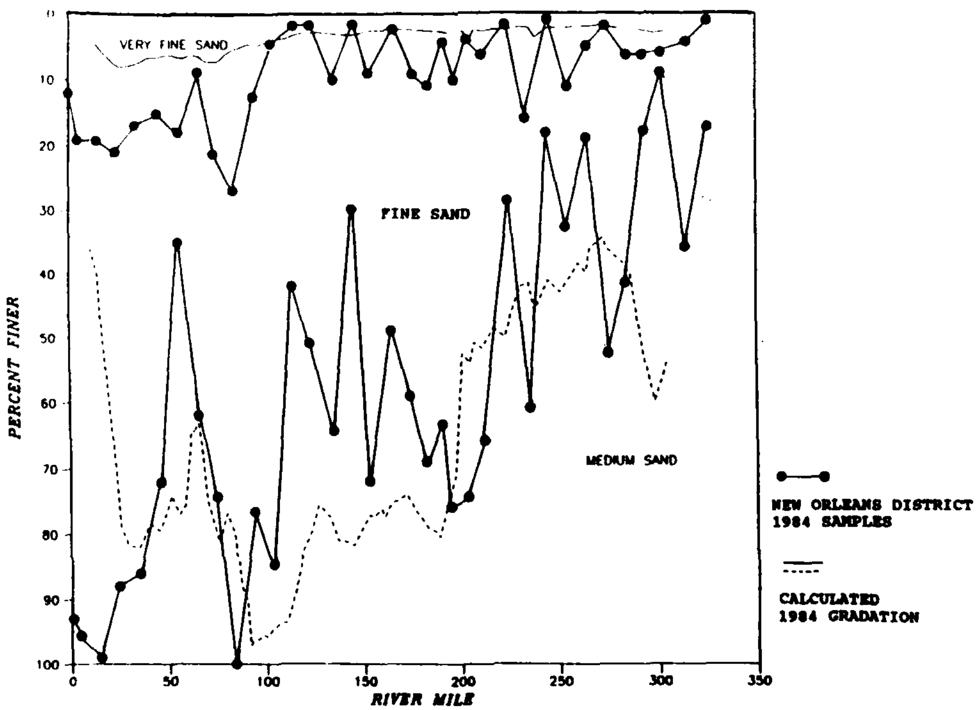

Figure 11. Comparison of 1984 calculated and sampled bed gradations

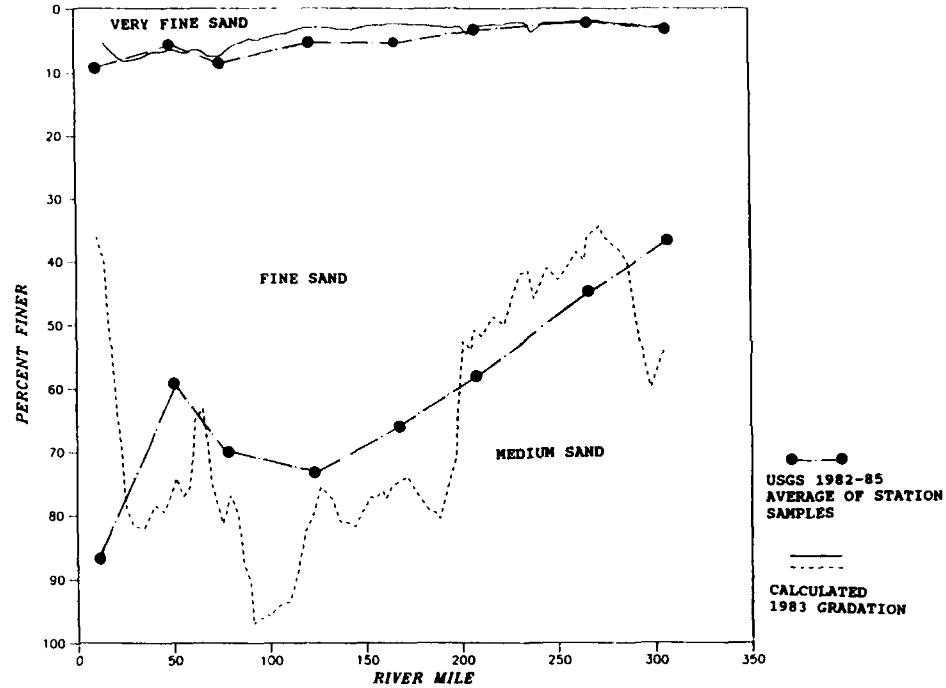

Figure 12. Comparison of 1983 calculated and 1982-85 sampled bed gradations

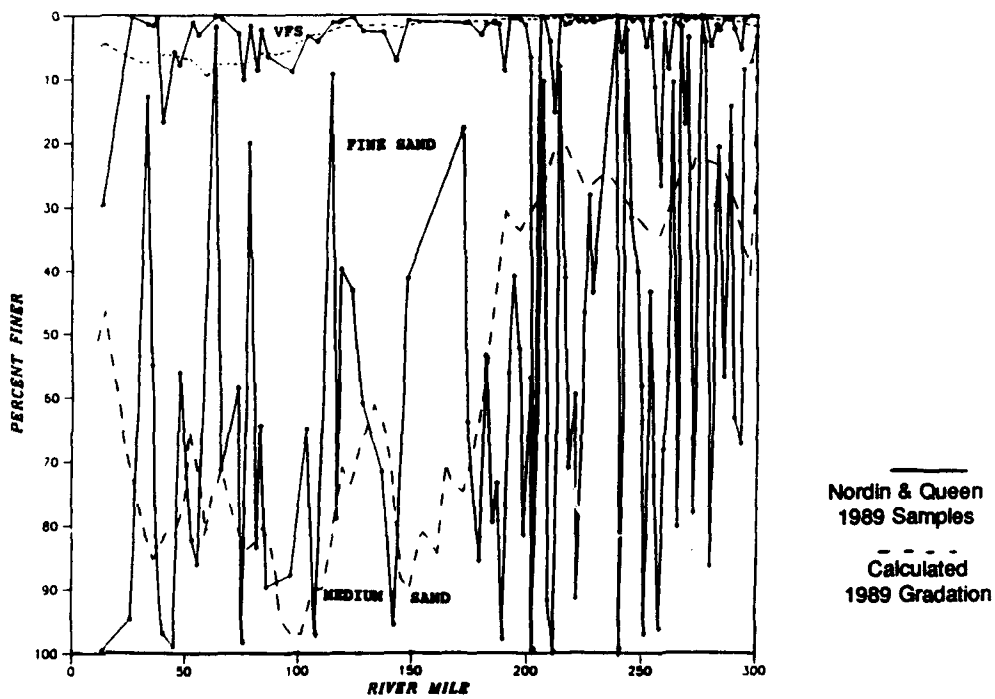

Figure 13. Comparison of 1989 calculated and sampled bed gradations

are known to vary laterally across the river, as well as in response to sinuosity, expansions, contractions, and obstructions. The numerical model calculates bed gradation based on steady, cross-sectional averaged hydraulic conditions. Calculated bed gradations from the model are expected to have some longitudinal variability attributable to departure of the model's cross sections from fully one-dimensional flow.

Adjustment to Dredging Volumes

52. The numerical model was adjusted to simulate the eight-year 1975-82 dredging total in Southwest Pass. Adjustment parameters are (a) the sediment concentrations leaving the river via the distributaries, (b) the bed shear stress threshold for the deposition of silt and clay, and (c) scour thresholds for the resuspension of silt and clays.

53. Concentrations of silt and clay in the distributaries and the river were set equal. This is considered reasonable because silt and clay are typically uniformly distributed in the water column. Distributary sand concentrations were assumed to be 50 percent of the river's sand concentration except at Head of Passes where Pass a Loutre and South Pass were assumed to have a sand concentration equal to 85 percent of the river's sand concentration. Lower sand concentrations in distributaries are justified because sand concentration is greater at the bottom of the water column and distributary outlets are typically at a higher elevation than the river channel. At Head of Passes the river bottom elevation is essentially equal in all three passes, but historical evidence indicates that a greater portion of sand moves into Southwest Pass. This extra material was set equal to the unmeasured load percentage (15 percent).

54. Historical dredging quantities in Southwest Pass were provided by the New Orleans District. It was estimated that 42 percent of the reported dredging occurs within the limits of the numerical model. The remainder is dredged downstream from East Jetty in the Gulf of Mexico. In the numerical model, deposition of silt and clay in Southwest Pass was adjusted by varying deposition and scour threshold coefficients in the pass until calculated dredging quantities duplicated reported quantities (Table 5). Varying these coefficients in Southwest Pass was deemed reasonable to account for the effects of salinity on sediment deposition.

55. In order to numerically simulate prototype dredging, it is also necessary to simulate the final prototype geometry in Southwest Pass. This was accomplished by setting the dredging template at each cross section such that after dredging the section would be similar to sections from the 1983 hydrographic survey which represented after-dredging conditions. This accounts for the 9.1 million cubic yards shown as initial dredging in Table 5.

Table 5

Calculated and Reported Dredging (Million Cubic Yards) in Southwest Pass*

Year		Calculated Dredging	Reported Dredging
Initial		9.1	-
1975		6.5	14.6
1976		3.4	5.8
1977		2.4	4.8
1978		4.2	5.0
1979		14.0	10.0
1980		5.6	5.4
1981		3.4	4.1
1982		5.6	4.7
	Total	54.2	54.4

* Head of Passes to East Jetty, assumed to be 42 percent of total reported quantities for Southwest Pass.

PART IV: STUDY RESULTS

Sediment Deposition Due to Operation of Bonnet Carre Spillway

56. During the early phases of the investigation, before the numerical model was adjusted, a flood discharge of 1,400,000 cfs for six weeks was anticipated on the Mississippi River at Tarbert Landing. Existing criteria called for operating the Bonnet Carre Spillway to maintain a discharge of 1,250,000 cfs at New Orleans. The numerical model was used to provide a qualitative evaluation of the effect of Bonnet Carre operations on sediment delivery to the delta passes.

57. This qualitative evaluation was made using a preliminary bed material gradation determined by the New Orleans District and by assuming that the river was carrying its full calculated sediment capacity based on that gradation. Only sand transport was considered. Transport of sand over the Bonnet Carre Spillway was considered to be negligible. A base test with a six-week discharge of 1,400,000 cfs followed by eight years of historical flows (1975-82) was used to compare flood operations with and without spillway operations.

58. The six-week flood of 1,400,000 cfs would deliver 18 million cubic yards of sand to the Mississippi River above that of the base test. With Bonnet Carre open, so that 1,250,000 cfs moved past New Orleans, much of this material would be deposited in the first 10 miles downstream from the spillway; however some deposition would occur throughout the study reach and 1.3 million cubic yards would be transported past Venice during the six-week flood. After five years, 10 million of the 18 million cubic yards would be transported past Venice with the remaining 8 million cubic yards slowly moving downstream at a rate determined by the magnitude of river discharges. Using the 1975-82 histograph, this rate would be about 3 miles/year. The predicted change in bed elevations due to the six-week flood at selected times during the eight-year simulation is shown in Plate 14.

59. With Bonnet Carre closed, so that the discharge past New Orleans was 1,400,000 cfs, there would be alternating sections of scour and deposition downstream from Bonnet Carre with scour being more prevalent. With Bonnet Carre closed, 18.8 million cubic yards above the base test would be carried past Venice during the six-week flood compared with 1.3 million cubic yards with the spillway open. During the next five years sand transport past Venice

would be 4.3 million cubic yards less. The predicted change in bed elevations due to the six-week flood at selected times during the eight-year simulation is shown in Plate 15.

60. Comparing the two alternative spillway operations, 17.5 million cubic yards more sand would pass Venice during the flood if Bonnet Carre was not operated (Table 6). After the flood, during the next five years, 12.9 million cubic yards less sand would pass Venice if the spillway was not operated. Eventually the same amount of material would pass Venice with either alternative. If Bonnet Carre was closed, most of the additional sand would pass Venice during the flood with the higher discharge available to help transport it through the Passes. If Bonnet Carre was opened, transport of the extra material past Venice would take several years to accomplish.

Table 6

Effect of Closing Bonnet Carre Spillway During a Six-week Flood of 1,400,000 cfs Followed by Eight Years of Historical Discharges

Time Period	Increase in Sediment Load Past Venice million cubic yards
Six-week flood	17.5
April-December 1975	-2.7
1976	-1.3
1977	-1.6
1978	-3.5
1979	-3.8
1980	-
1981	-
1982	0.1

Projected Changes in Bed Volume

61. Whereas, the ability of the numerical model to quantitatively predict long-term changes in bed elevations at specific locations remains unconfirmed, the model is considered reliable for assessing relative changes due to changes in boundary conditions, such as sediment inflow. The historical

simulation was extended to 1989, and cumulative bed volume changes were calculated using both the 1976-82 sediment inflow curves for the entire simulation and the new sediment inflow curves for 1983-85 and 1986-89. The calculated cumulative volumes through 1989 are shown in Figure 14. When the 1976-82 sediment inflow curves were used for the entire simulation, sediment accumulation of about 48 million cubic yards occurred in the study reach between 1984 and 1989. Almost all of the material was deposited between river miles 30 and 85. Upstream from mile 100 the bed changes were less significant, although a noticeable degradation trend was indicated upstream from river mile 270. Using the reported 1983-89 sediment inflow curves resulted in 45 million cubic yards of degradation in the 300-mile reach for the five-year period. This is a 93-million-cubic yard difference from the results using the 1976-82 sediment inflow curves. Even with calculated net degradation, using the 1983-89 sediment inflow curves, aggradation of about 40 million cubic yards was calculated between river miles 30 and 85. Upstream from river mile 85, calculations with the 1983-89 sediment inflow curves indicate a general degradation trend increasing toward the upstream boundary at Tarbert Landing. The most significant degradation occurred upstream from river mile 270.

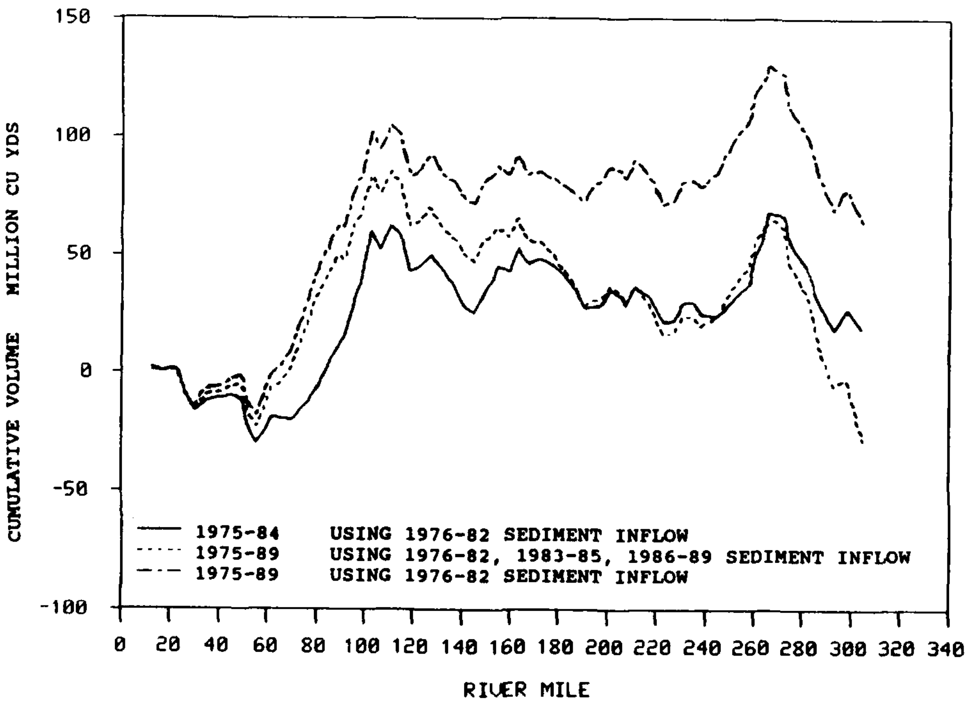

Figure 14. Calculated cumulative bed volume changes, 1975-89

62. Numerical model results indicated that 147 million tons of sand were transported past Tarbert Landing between 1984 and 1989 when 1976-82 sediment inflow curves were used, and 27 million tons when 1983-85 and 1986-89 sediment inflow curves were used. However, sand transport past Venice was essentially the same with either sediment inflow assumption, with 87 million tons passing Venice during the five-year period with the 1976-82 sediment inflow curves and 84 million tons with the 1983-85 and 1986-89 sediment inflow curves.

63. Numerical model results indicate that the effects from changes in sediment inflow would primarily be felt in the reach just downstream from Tarbert Landing to river mile 270. The model indicated degradation in this reach regardless of the sediment inflow assumption. An aggradation trend is apparent between river miles 30 and 85 with either of the sediment inflow assumptions.

Projected Behavior of Sediment Sill

64. The numerical model was used to evaluate a proposed sediment sill which would be used to retard saltwater intrusion during low flows. This evaluation was part of a larger study which investigated the impact of salinity intrusion with a proposal to increase the navigation channel depth to 55 ft (Johnson, Boyd, and Keulegan 1987). The sill was to be constructed at river mile 63.4 from bed material dredged from upstream. It was to have a generally triangular shape with 1V:40H side slopes and a crest elevation of -55 ft.* The numerical model was used to evaluate the stability of the proposed sill during high flows and its effect on water-surface elevation.

65. Steady state discharges ranging between 200,000 and 800,000 cfs were simulated for a period of 150 days; the following erosion occurred off the top of the sill:

Discharge, cfs	Erosion, ft
200,000	0.02
300,000	1.4
400,000	2.0
450,000	5.0

* Elevations are in feet referenced to National Geodetic Vertical Datum (NGVD).

Rates of erosion for higher discharges are shown in Figure 15. At a discharge of 800,000 cfs the sill washed out in about five days. The sill was considered stable with a discharge less than 400,000 cfs.

66. The effect of the proposed sill on water-surface elevations was determined by comparing results from computer runs with and without the sill in place. As shown in Figure 16, the sill had no significant effect on water-surface elevations.

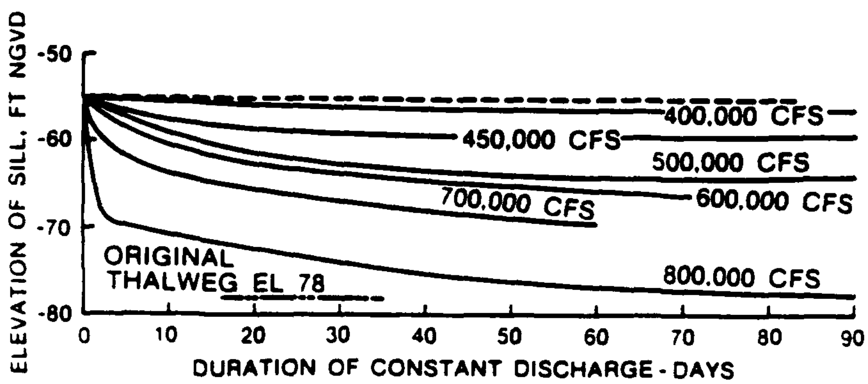

Figure 15. Predicted erosion of sediment sill at river mile 63.4

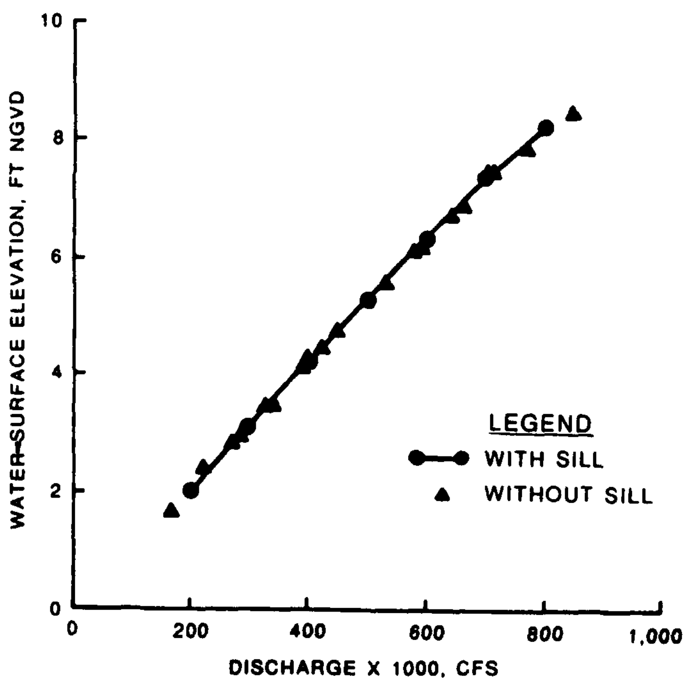

Figure 16. Effect of sediment sill on water-surface elevation at river mile 63.7

Model Evaluation Using Prototype Sediment Sill

67. During the 1988 low water season, the sediment sill was actually constructed across the Mississippi River at mile 63.4. The dimensions of the actual sill were slightly different from the tested sill. The actual sill had a crest elevation of -45 ft NGVD instead of -55 ft NGVD. The sill remained in place throughout the low flow season and washed out on the rise of the 1989 annual hydrograph. Several hydrographic surveys were taken in the vicinity of river mile 63.4 while the sill was in place and during the washout. These data provided an excellent opportunity to evaluate the ability of the numerical model to simulate the washout process.

68. Four cross sections were added to the numerical model to define the sill. The new cross sections were 400 ft apart: one at the original sill crest at river mile 63.4, one upstream, and two downstream. Cross sections from the 1973-75 hydrographic survey were used as initial geometry for the rest of the model. Initial bed material gradations at the new cross sections were interpolated from the cross sections in the base model. Sediment inflow from the 1985-89 sediment rating curves was taken to be representative; however, sediment inflow developed using the 1976-82 sampled data at Tarbert Landing was used for a sensitivity test.

69. The average bed elevation in the 750-ft-wide navigation channel at river mile 63.4 was determined from each of the six hydrographic surveys, which spanned the period between 31 Jul 1988 and 13 Feb 1989. The same time period was simulated in the numerical model. Calculated and surveyed average bed elevations at river mile 63.4 are compared in Figure 17. There is good agreement between the computed and surveyed data; both show that the sill remained stable between July and November when the discharge varied between about 120,000 and 150,000 cfs. By 5 Dec 1988, when the next survey was taken, the discharge had risen to about 600,000 cfs and the sill had begun to wash out. After 5 Dec, the hydrograph began to fall and the numerical model indicated that erosion of the sill ceased. Erosion recommenced by 9 Jan 1989, when the discharge had increased to about 650,000 cfs. At the end of the simulation, on 13 Feb 1989, the sill was essentially washed out at river mile 63.4 in both the calculated and surveyed sections.

70. In the prototype, the sill crest moved downstream, between December 1988 and February 1989, as the sill washed out. Therefore, the maximum crest

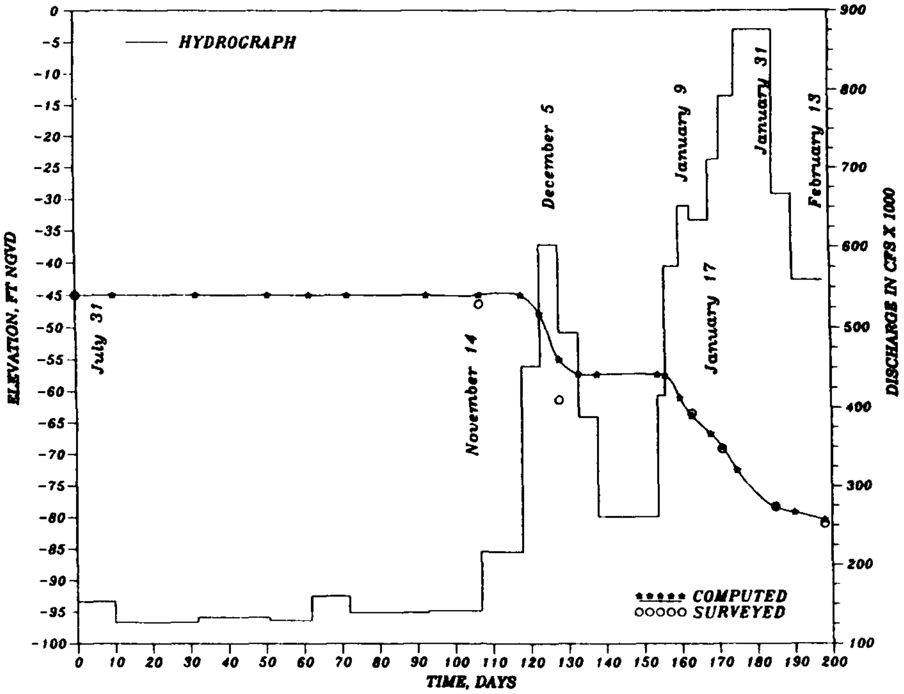

Figure 17. Comparison of average bed elevations at river mile 63.4 between 31 Jul 1988 and 13 Feb 1989

elevation did not occur at river mile 63.4 once the washout process began. However, in the model, the crest was located at mile 63.4 for the most of the simulation. Calculated and surveyed sill crest elevations are compared in Figure 18 which shows higher crest elevations in the prototype after 9 Jan 1989. This effect is further demonstrated with comparisons of cumulative changes in cross-sectional areas at mile 63.4 in Plate 16. The prototype shows a more rapid rate of decline due to movement of the sill downstream. The effect of this downstream progression of the sill is also apparent in Plates 17 and 18, which compare calculated and surveyed cumulative change in area at the two cross sections downstream from mile 63.4. The sill crest passes the first section, 400 ft downstream from 63.4, after 170 days, and the second section, 800 ft downstream from 63.4, after 185 days. Comparisons of calculated and surveyed profiles of the average bed elevation in the navigation channel are plotted in Plate 19. Calculated results from the numerical model reproduced the correct lowering of the bed elevation at river mile 63.4, but did not simulate the movement of the sill downstream and, therefore, overestimate the rate of sill crest erosion. This effect is attributed to the lack of an algorithm in TABS-1 for calculation of downstream migration of bed

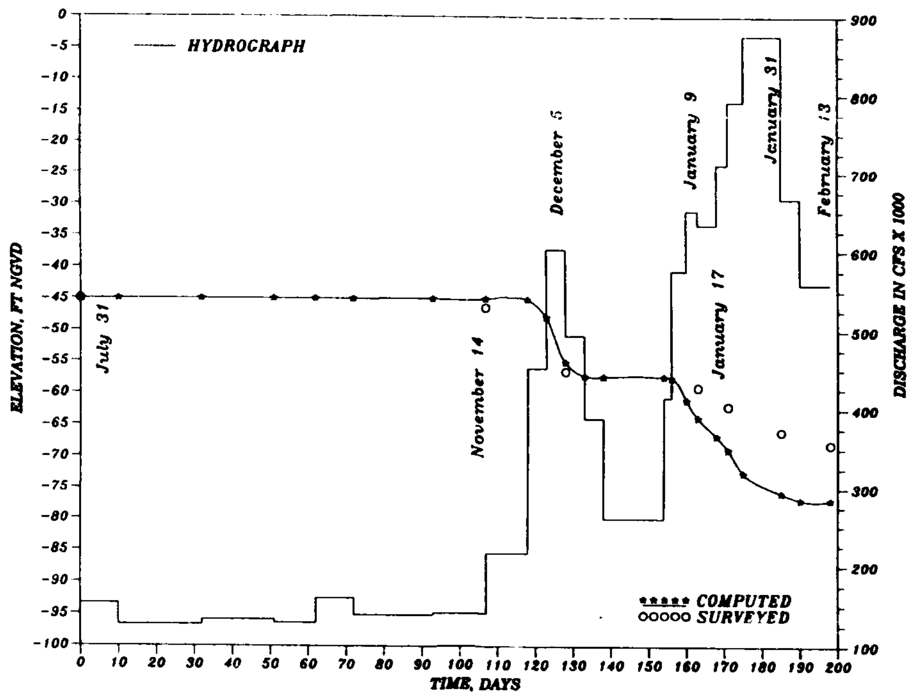

Figure 18. Comparison of sill crest elevations between 31 Jul 1988 and 13 Feb 1989

forms. Bed-form migration is primarily a function of bed-load transport which has significant temporal variation. TABS-1 simulates the <u>average</u> bed-load transport rate and therefore does not reproduce the bed forms created by the variable bed-load transport rate.

71. In order to test the sensitivity of the model results to sediment inflow, the 1976-82 sediment inflow curves were substituted into the model and the simulation was repeated. The higher concentration sediment inflow curves did not significantly affect the results. This is attributed to the relatively short time frame for washing out of the sill and to the distance between the sediment inflow boundary and the sill.

72. The numerical model provided a generally accurate simulation of the washout of the sediment sill. It is significant that no additional adjustment to the numerical model was required (aside from adding additional cross sections). This establishes confidence for using the model in the predictive mode in similar future studies.

73. The numerical model simulates both suspended and bed-load transport. However, it does not have an analytical algorithm for calculating downstream

migration of bed forms and, therefore, did not reproduce the downstream movement of the sediment sill.

Diversion at Bonnet Carre

74. A proposed diversion was located at Bonnet Carre, where up to 30,000 cfs would be diverted. The effect of sediment diversion on deposition and dredging in Southwest Pass was determined with the numerical model. For purposes of this study, the sediment concentrations of sand, silt, and clay were considered equal in the diversion and the river. A 32-year period was tested by running the 1975-82 hydrograph four times. Calculated dredging and deposition quantities were compared with results from a base test which was run for the same period without diversion. A constant diversion of 30,000 cfs was assumed for the model.

75. The results indicate that the diversion had only a slight effect on dredging in Southwest Pass. During the first 12 years, dredging requirements were 5 percent less with the diversion. This amounted to an average annual decrease of about 300,000 cubic yards. After the twelfth year, the general trend of dredging was greater with the diversion. The average annual increase was about 100,000 cubic yards. Calculated dredging quantities with and without the diversion are compared in Table 7. The initial 9.1 million cubic yards of dredging that occurs with both conditions represents the dredging required to establish the designated navigation channel. The total dredging required during the 32-year simulation is essentially the same with or without the diversion. The effect of the diversion on accumulated dredging as the simulation progressed is shown in Figure 19. This figure shows the accumulated differences in dredging volume. For example, after 12 years, there was a total benefit of 3 million cubic yards with the diversion scheme. Also, demonstrated with this figure is the significant effect that the annual hydrograph has on dredging requirements.

76. The effect of the diversion scheme on total deposition in Southwest Pass can be evaluated by comparing the sums of dredging and accumulated deposition. Deposition occurs in Southwest Pass below and on either side of the designated navigation channel. This comparison is also shown in Figure 19.

77. The diversion scheme will also have an effect on deposition upstream from Southwest Pass. Sediment deposition upstream from Head of Passes will

Table 7

Effect of Bonnet Carre Diversion

		Dredging* 10^6 yd^3			Deposition in Southwest Pass 10^6 yd^3		Accumulated Vol River Mile 140 10^6 yd^3		
Year	Hydrograph	Without Diversion	With Diversion	Difference Δ	Without Diversion	With Diversion	Without Diversion	With Diversion	Difference Δ
0		9.10	9.10						
1	1975	6.53	6.04	-0.49	-6.98	-6.95	13.63	15.79	2.16
2	1976	3.37	3.39	+0.02	-4.90	-4.83	21.05	24.60	3.55
3	1977	2.37	2.24	-0.13	-3.39	-3.32	29.68	34.33	4.65
4	1978	4.25	4.06	-0.19	-2.37	-2.60	40.54	47.32	6.78
5	1979	14.06	12.33	-1.73	1.93	1.96	73.38	83.06	9.68
6	1980	5.63	5.71	+0.08	1.86	2.54	85.66	95.66	10.00
7	1981	3.41	3.42	+0.01	2.60	3.26	94.47	105.19	10.72
8	1982	5.62	5.66	+0.04	1.68	2.93	109.96	120.68	10.72
9	1975	5.19	5.26	+0.07	2.06	1.71	135.11	147.39	12.28
10	1976	2.70	2.56	-0.14	3.39	2.90	141.92	155.88	13.96
11	1977	2.23	2.05	-0.18	3.97	3.41	149.74	165.06	15.32
12	1978	3.32	2.90	-0.42	4.85	4.18	160.26	178.36	18.10
13	1979	11.49	12.42	+0.93	7.35	7.99	198.48	212.34	13.86
14	1980	4.36	4.99	+0.63	7.50	8.26	211.46	225.19	13.73
15	1981	3.00	3.03	+0.03	8.16	8.98	220.00	234.56	14.56
16	1982	4.99	5.09	+0.10	7.84	8.62	235.44	249.97	14.53
17	1975	4.88	3.69	-1.19	8.65	8.82	260.38	276.96	16.58
18	1976	2.64	2.64	-	9.43	9.30	266.94	284.83	17.89
19	1977	2.17	1.80	-0.37	9.85	9.64	274.63	293.78	19.15
20	1978	3.29	3.12	-0.17	10.72	10.18	284.60	306.22	21.62
21	1979	11.98	13.90	+1.92	12.03	12.49	322.22	337.09	14.87
22	1980	4.24	4.61	+0.37	12.15	12.81	334.97	350.09	15.12
23	1981	2.74	2.96	+0.22	12.88	13.39	343.32	359.13	15.81
24	1982	4.77	4.99	+0.22	12.48	13.12	358.04	374.07	16.03
25	1975	4.55	3.88	-0.67	13.02	13.38	382.42	400.33	17.91
26	1976	2.45	2.44	-0.01	13.77	13.74	388.93	408.03	19.10

(Continued)

* East Jetty to Head of Passes.

Table 7. (Concluded)

		Dredging 10^6 yd^3			Deposition in Southwest Pass 10^6 yd^3		Accumulated Vol River Mile 140 10^6 yd^3		
Year	Hydrograph	Without Diversion	With Diversion	Difference Δ	Without Diversion	With Diversion	Without Diversion	With Diversion	Difference Δ
27	1977	2.09	1.66	-0.43	14.03	13.99	396.26	416.67	20.41
28	1978	3.54	2.86	-0.68	14.76	14.73	405.64	428.80	23.16
29	1979	11.95	13.96	+2.01	16.02	16.43	443.92	459.55	15.63
30	1980	4.24	4.26	+0.02	16.19	16.72	456.34	472.66	16.32
31	1981	2.81	2.89	+0.07	16.81	17.24	464.09	481.36	17.27
32	1982	4.70	4.85	+0.15	16.38	16.94	478.40	495.88	17.48
	Total	164.66	164.76						
	Average	5.15	5.15	0.00					

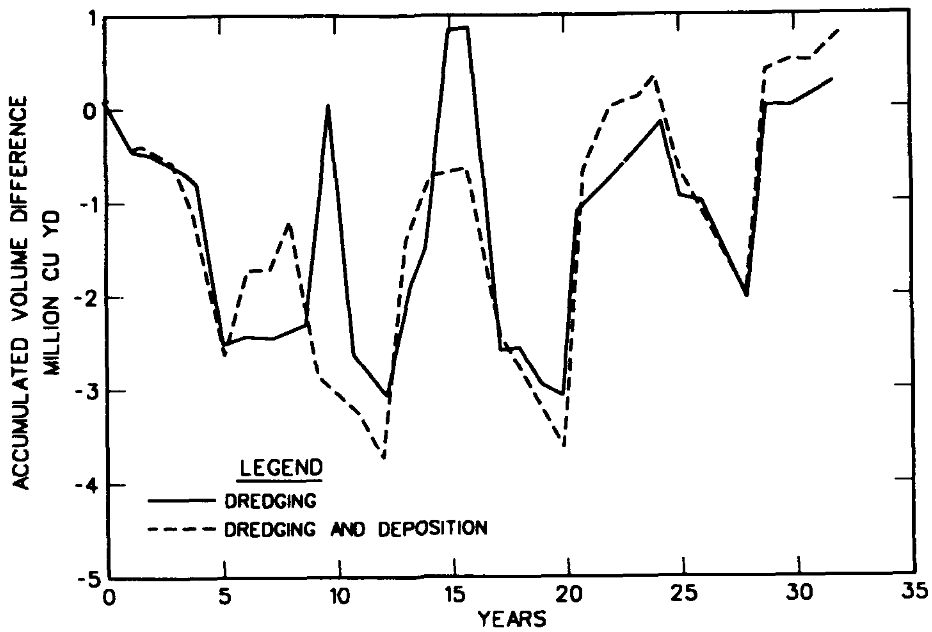

Figure 19. Difference in Southwest Pass accumulated volumes due to diversion at Bonnet Carre

increase by approximately 1.5 million cubic yards per year for the first 12 years. The increase levels off to less than 200,000 cubic yards per year after 12 years. The accumulated increase in sediment deposition in the Mississippi River between East Jetty and Bonnet Carre due to the operation of the diversion is shown in Figure 20.

Diversion at River Mile 6.7

78. The numerical model was used to evaluate changes in dredging and deposition quantities that would result from a constant 10 percent diversion of water and varying concentrations of sediment at river mile 6.7. The model was tested with three different sand concentrations specified in the diverted flow: (a) no sand diverted; (b) a sand concentration of 50 percent of that upstream of the diversion, and (c) a sand concentration of 100 percent of that upstream of the diversion.* An eight-year hydrograph (1975-82) was repeated

* In all tests silt and clay concentrations were considered equal in the river and the diversion.

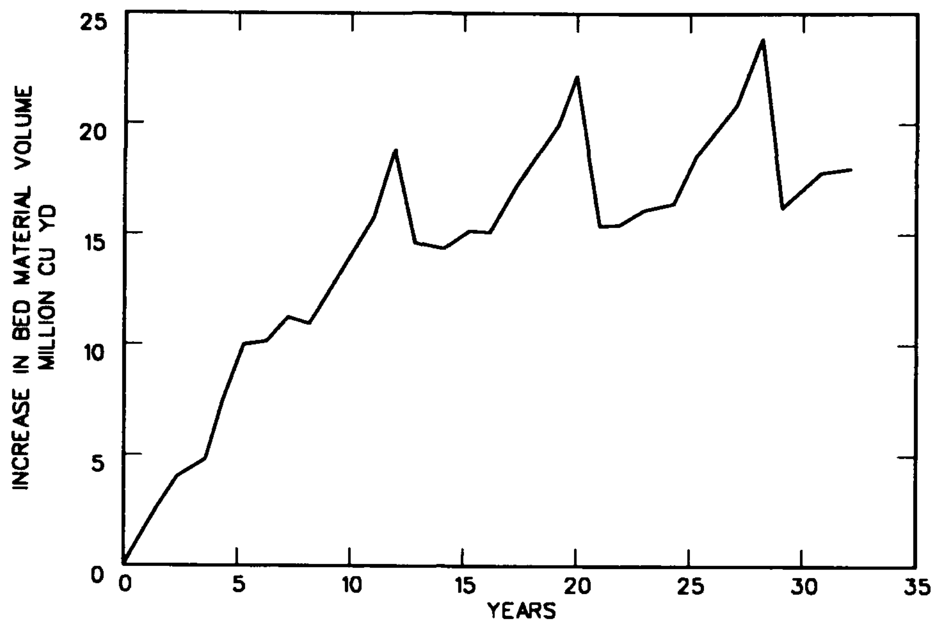

Figure 20. Effect of Bonnet Carre diversion on deposition between East Jetty and Bonnet Carre

four times to simulate a 32-year record for comparison of dredging quantities. Cumulative dredging quantities are shown in Figure 21. Since some sediment will deposit outside the dredged channel, dredging quantities do not account for all the deposition along Southwest Pass. The total effect of the diversion on deposition is shown in Figure 22 as the accumulation of dredging plus deposition outside of the dredged channel limits. Results are summarized in Table 8.

79. The results indicate that the diversion would cause an increase in mean annual dredging of between 440,000 and 870,000 cubic yards, or 8 and 16 percent, respectively, depending on the concentration of sand in the diversion. This increase is primarily due to the loss of sediment-transport capability because of the decrease in water discharge through Southwest Pass.

Major Diversion Schemes

80. The numerical model was used to evaluate changes in dredging and deposition quantities that would result from a major flow diversion of the

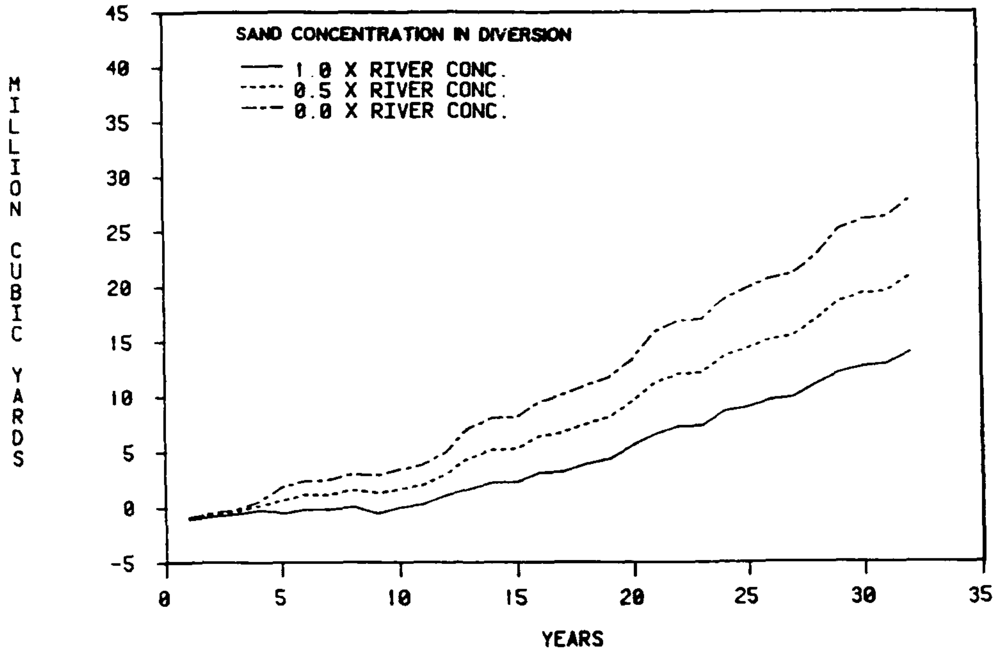

Figure 21. Effect of 10 percent flow diversion at river mile 6.7 on dredging in Southwest Pass

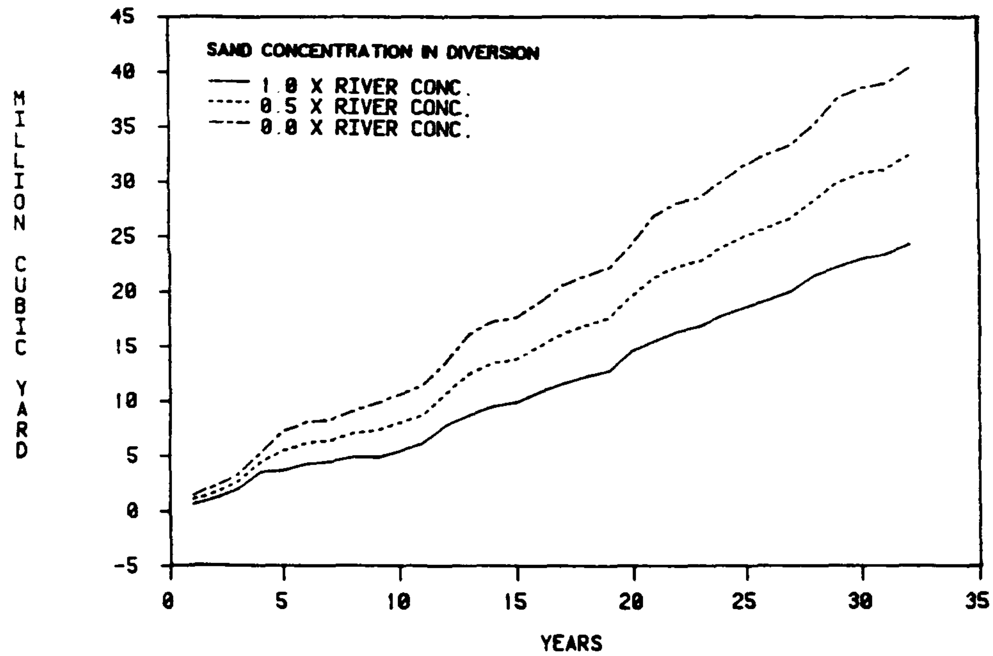

Figure 22. Effect of 10 percent flow diversion at river mile 6.7 on dredging and deposition in Southwest Pass

Table 8

Effect of 10 Percent Diversion at River Mile 6.7

Supplement II Conditions

Sand Concentration in Diversion as Percent of River Concentration	Increase in Mean Annual Dredging*		Increase in Mean Annual Dredging and Deposition*	
	$yd^3 \times 10^6$	Percent	$yd^3 \times 10^6$	Percent
100	0.44	8.1	0.76	13.0
50	0.65	12.0	1.01	17.2
0	0.87	16.0	1.26	21.5

* Based on 1975-82 hydrograph repeated four times for 32-year period of record.

Mississippi River. Three diversion locations were evaluated. These were: East Caernarvon (river mile 81), Myrtle Grove (river mile 59), and Bohemia Spillway (river mile 44). For each location tested 70 percent of the flow in the Mississippi River was diverted for all discharges. A range of conditions were tested with respect to the sand concentration in the diverted flow. The concentration of silts and clays was assumed to be the same in the river and the diversion. Two assumptions with respect to downstream geometry were evaluated: (a) Supplement II (USACE-NOD 1984) conditions and (b) a plan that called for closure of all secondary passes, and raised overbank elevations to contain river flows in the Mississippi River and Southwest Pass from the diversion to East Jetty. For each location and downstream channel condition, three different assumptions for sand concentration in the diversion were tested. In all, 18 different combinations were tested and compared with a base test. The base test was a model of Supplement II conditions. In the model, dredged material was removed from the river.

81. The model was tested with three different sand concentrations in the diverted flow. The most severe deposition in the Mississippi River downstream from the diversions occurred when no sand was diverted. This is an extreme scenario and can be considered as an upper limit for possible dredging and deposition. Diversion of sand concentrations that were 50 and 100 percent of the sand concentration upstream in the river were also tested. These two scenarios are within the range of probable occurrence. If the diversion were to be located at the lee end of a point bar, it is possible that

concentrations in the diversion could be higher than in the upstream river; however, this scenario was not tested.

82. An eight-year hydrograph (1975-82) was repeated four times to simulate a 32-year period for comparing dredging plus deposition.

83. Calculated water-surface elevations upstream from the diversions were lower than for existing conditions without the diversion because of the reduced flow in Southwest Pass and in the main stem of the Mississippi River below the diversion. This result is dependent on the assumption that backwater control up the main channel determines the water surface at the diversion and that the diversion channel will form to accommodate this water surface. The effect of the lower water surface elevation is increased degradation (or less aggradation) upstream from the diversion. As a result, sediment concentrations will increase and the potential for downstream deposition will be greater. Also the net accumulated deposition in the study reach will tend to be reduced.

84. The results indicate that the diversions caused a significant increase in net dredging in the river immediately downstream from the diversion where most of the sediment deposited. With existing river conditions downstream from the diversion, increases in net dredging ranged between 35 and 320 percent, depending on the diversion's location and assumed sand concentration. Increases in the sum of dredging and deposition ranged between 17 and 106 percent. When all the passes and outlets were closed, changes in net dredging ranged from a reduction of 3 percent to an increase of 280 percent. For this condition, dredging in Southwest Pass was essentially eliminated. Increases in the sum of dredging and deposition ranged between 0 and 83 percent. Accumulated differences between calculated dredging with and without the diversions for Supplement II conditions are shown in Plates 20-22 and those for closed passes in Plates 23-25. Accumulated differences between the sum of dredging and deposition for existing downstream conditions are shown in Plates 26-28 and for closed passes in Plates 29-31. These plates demonstrate the significance of the assumed sand concentration in the diversion. Average annual dredging quantities were estimated by dividing the calculated accumulated dredging and deposition differences at the end of the simulation by the 32-year test period. These are shown for each of the three diversion sites tested in Tables 9-11. A comparison of these tables shows that the effect of the diversion site is not as significant as sand concentration on annual

Table 9

Flow Diverted at East Caernarvon (Mile 81)

Sand Concentration in Diversion as Percent of River Concentration	Increase in Mean Annual Dredging*				Increase in Mean Annual Dredging and Deposition*	
	Southwest Pass		East Jetty to Reserve		East Jetty to Reserve	
	$yd^3 \times 10^6$	%	$yd^3 \times 10^6$	%	$yd^3 \times 10^6$	%
	Supplement II					
100	−4.6	−65	6.4	86	3.7	17
50	−4.6	−65	15.3	205	12.9	60
0	−4.6	−65	23.8	320	22.0	106
	Secondary Passes Closed					
100	−7.1	−99	3.8	51	0.0	0
50	−7.1	−99	12.5	168	9.0	42
0	−7.1	−65	20.8	280	17.8	83

* Based on the 1975–82 hydrograph repeated four times for 32-year period of record.

Table 10

Flow Diverted at Myrtle Grove (Mile 59)

Sand Concentration in Diversion as Percent of River Concentration	Increase in Mean Annual Dredging*				Increase in Mean Annual Dredging and Deposition*	
	Southwest Pass		East Jetty to Reserve		East Jetty to Reserve	
	$yd^3 \times 10^6$	%	$yd^3 \times 10^6$	%	$yd^3 \times 10^6$	%
	Supplement II					
100	−4.4	−63	2.6	35	5.0	23
50	−4.4	−63	8.0	108	12.3	57
0	−4.4	−63	12.8	172	19.0	88
	Secondary Passes Closed					
100	−7.1	−99	−0.3	−3	1.2	6
50	−7.1	−99	5.1	68	8.1	37
0	−7.1	−99	10.0	134	14.8	69

* Based on the 1975–82 hydrograph repeated four times for 32-year period of record.

Table 11

Flow Diverted at Bohemia Spillway (Mile 44)

Sand Concentration in Diversion as Percent of River Concentration	Increase in Mean Annual Dredging* Southwest Pass $yd^3 \times 10^6$	Southwest Pass %	East Jetty to Reserve $yd^3 \times 10^6$	East Jetty to Reserve %	Increase in Mean Annual Dredging and Deposition* East Jetty to Reserve $yd^3 \times 10^6$	East Jetty to Reserve %
			Supplement II			
100	−4.2	−60	3.7	49	8.0	37
50	−4.2	−60	9.0	121	15.1	70
0	−4.2	−60	13.6	183	21.9	101
			Secondary Passes Closed			
100	−7.1	−99	−0.8	10	3.9	18
50	−7.1	−99	6.1	80	10.7	49
0	−7.1	−99	10.4	140	17.1	79

* Based on the 1975–82 hydrograph repeated four times for 32-year period of record.

dredging. The benefit from closing the passes and outlets can be determined by comparing the increase in mean annual dredging for the two downstream conditions in the tables. The result is about 3 million cubic yards per year and is essentially independent of the diversion's location or sand concentration.

85. In general, the effect of diverting water and sediment will be increased deposition and dredging downstream because the reduced discharge will not be able to maintain the existing sediment concentration. This effect will be reduced if the secondary outlets are closed, because sediment transport potential will not be further reduced by distributary outflow. Thus dredging in Southwest Pass is reduced or eliminated.

Application to Redeye Crossing

86. Redeye Crossing, at river mile 224, experiences significant shoaling problems and dredging annually. The navigation channel in this reach is maintained at a width of 500 ft and a depth of 40 ft below low water reference plane (LWRP); LWRP is at el 2.3 ft NGVD. The numerical model was adapted to simulate the existing dredging operation at Redeye Crossing and then to

evaluate the effect of flow constriction on dredging requirements with a 45-ft draft project in place.

87. A new dredging algorithm was incorporated into the TABS-1 model to better simulate the dredging operation at Redeye Crossing. With the new dredging routine, both dredging capacity and initiation time of operations are specified. Dredging calculations begin at the downstream control volume in the dredging reach and continue until the specified bed elevation is reached. If the specified bed elevation is not reached during a computational time-step, then normal hydraulic and sediment calculations are made for the subsequent time-step and dredging calculations continue. Dredged material is returned to the water column in the next downstream control volume. When specified dredging is complete, calculations proceed to the next upstream control volume and begin dredging calculations at that section. In a like manner, calculations proceeded in an upstream direction through the dredging reach. The advantage of this new algorithm is that dredged material is returned to the river in a manner similar to prototype operations and dredging rates can be specified.

88. The numerical model was shortened for this analysis. The downstream boundary was set at Donaldsonville, at river mile 175, which is the first stage-discharge gage below Redeye Crossing. Downstream water-surface elevations were based on an average 1983 stage-discharge rating curve at Donaldsonville. The upstream boundary remained at Tarbert Landing. Sand inflow was determined from average sediment inflow rating curves based on 1975-89 sampled data at Tarbert Landing. Model geometry was based on the 1973-75 hydrographic survey. Additional cross sections were added to the numerical model to provide additional resolution in the vicinity of Redeye Crossing. Cross sections were about 2,000 ft apart in the crossing (Plate 32). Roughness coefficients were adjusted such that calculated water-surface elevations matched average stages at Baton Rouge. This adjustment was necessary due to the addition of cross sections in the Redeye reach.

89. Annual histographs were developed for the years 1975-89. These histographs were used to obtain annual dredging quantities for a range of hydrologic conditions: 1979 and 1983 were high runoff years; 1976, 1977, and 1981 were low runoff years.

90. The numerical model was used to calculate dredging quantities between 1975 and 1989 at Redeye Crossing. In these tests, the dredging prism

was taken from a September 1988 post-dredging survey. In the numerical model, dredging operations were specified to occur on the recession limb of the annual hydrograph. A dredging capacity of 40,000 cubic yards per day was assumed, and two upstream sweeps were calculated each year. Calculated quantities are not directly comparable to reported quantities because the initial bed geometry and the exact limits of dredging are unknown. The purpose of the comparison was to determine if calculated results are reasonable. Model and prototype calculations are compared in Table 12. Total calculated dredging over the 15-year period was within 5 percent of reported dredging.

Table 12

Comparison of Reported and Calculated Annual Dredging, Million Cubic Yards

Annual Hydrograph	Reported Dredging	Calculated* Dredging
1975	4.88	3.23
1976	1.07	0.71
1977	1.75	0.81
1978	2.67	2.42
1979	1.21	2.40
1980	3.59	2.14
1981	0.51	1.00
1982	2.66	2.17
1983	1.56	2.38
1984	——	2.35**
1985	1.02	2.76
1986	1.01	1.66
1987	——	0.95**
1988	1.65	0.75
1989	——	1.56**
Total	23.58	22.43

* Based on 40,000 cubic yards per day dredging capacity and two upstream sweeps per year on recession of annual hydrograph.

** Not included in total because reported dredging volumes were not available.

91. The numerical model was used to evaluate the effect of flow constriction on dredging requirements in Redeye Crossing. The 1975-89 hydrograph was used in the evaluation. The same dredging scenario assumed in the historical simulation was assumed to test the dike design. The TABS-1 numerical

model is a one-dimensional model that uses average cross-sectional hydraulic parameters. The effect of localized velocity accelerations, secondary currents, and eddies are unaccounted for. Lateral expansion and contraction of flow between dikes is not calculated and must be accounted for by geometric input to the numerical model. This makes the evaluation of dike fields difficult. The approach taken in this study was to use a projection of the dike field throughout the crossing. Effectively, this assumes complete filling to the dike crest between dikes. More detailed two-dimensional numerical and physical model studies are required to properly account for these effects. Results of the one-dimensional numerical model should be considered approximate.

92. The initial plan was to place submerged dikes alternating on both the right and left descending banks as shown in Plate 32. The tops of the dikes would be at el -12.0, allowing for 15.5 ft of clearance above LWRP. This plan was successful in reducing total dredging in the crossing by about 73 percent. However, dredging was still required during high flow years. During the 1979 flood peak, a maximum of about 14 ft of scour was calculated in Redeye Crossing due to constriction by the dike field. During the 1983 flood peak, a maximum scour of about 24 ft was calculated. The one-dimensional model does not account for additional local scour that would be expected at the dike toes.

93. A trace width of 3,000 ft was tested by increasing the dike crests adjacent to the banks to el 30.0 ft. With this plan dredging was reduced 89 percent. Maximum calculated scour was 19 ft in 1979 and 25 ft in 1983.

94. A second plan consisting of seven dikes with crests at el -12.0 on the right descending bank and at el -15.0 on the left descending bank was tested (Plate 33). Although there were fewer dikes in this plan, more constriction was provided. This plan reduced dredging 92 percent during the 15-year simulation. However, extensive scour was calculated through the reach, with a maximum of 42 ft in 1979 and 30 ft in 1983.

95. Annual calculated dredging quantities for the three plans tested are shown in Table 13.

96. Dredging requirements can be significantly reduced in Redeye Crossing by constricting the flow with a dike field. The dike fields tested did not eliminate the need for dredging, especially during high flow years. The dike plans tested introduced significant general scour in the crossing at high

Table 13

Calculated Annual Dredging,* Million Cubic Yards

Annual Hydrograph	Existing Conditions	Plan 1 Design	Plan 1 with 3,000-ft Trace Width	Plan 2 Design
1975	3.23	0.63	0.00	0.27
1976	0.71	0.01	0.71	0.00
1977	0.81	0.01	0.00	0.00
1978	2.42	0.85	0.06	0.14
1979	2.40	0.86	0.38	0.12
1980	2.14	0.45	0.24	0.03
1981	1.00	0.04	0.05	0.00
1982	2.17	0.62	0.24	0.02
1983	2.38	1.45	0.46	0.47
1984	2.35	0.54	0.30	0.27
1985	2.76	0.52	0.19	0.11
1986	1.66	0.32	0.07	0.08
1987	0.95	0.36	0.29	0.21
1988	0.75	0.28	0.20	0.12
1989	1.56	0.46	0.41	0.33
Total	27.29	7.40	2.89	2.17
Percent of existing		27	11	8

* Based on dredging capacity of 40,000 cubic yards per day and two upstream sweeps per year on recession of annual hydrograph.

flow. Additional local scour (not simulated in the one-dimensional model) would occur at the dike toes. Questions related to dike spacing and alignment are influenced by two- and three-dimensional flow phenomena and require more extensive numerical and physical modeling studies. Final design for the dike field at Redeye Crossing was to be determined using a two-dimensional numerical model (TABS-2) and a movable-bed physical model.

PART V: CONCLUSIONS

97. The numerical model, adjusted to simulate cumulative bed volume changes between the 1961-63 and 1973-75 hydrographic surveys, was used to simulate the historical period between the 1973-75 and 1983-85 hydrographic surveys. In terms of net change in bed volume in the 300-mile reach, the calculated results from the hydrographic survey and the numerical model were similar. However, the calculated longitudinal distributions of aggradation and degradation from the numerical model and the hydrographic surveys were dissimilar. This difference is attributed to annual variations in prototype bed elevations, which appear to be more significant than any long-term aggradation or degradation trend. These annual variations are associated with the rise and fall of annual hydrographs and with dredging. The hydrographic surveys were taken over two-year periods at various points on the annual hydrographs. Generally, data from the two surveys, at specific cross sections, were not homologous. Due to this inconsistency, it was not possible to use the hydrographic survey data to verify the long-term predictive capability of the numerical model. However, based on successful duplication of long-term dredging records and measured sediment transport, it has been concluded that the numerical model is appropriate for evaluations of dredging scenarios; the effects of specified flow diversions on sedimentation downstream; bed response to changes in discharge, sediment concentration, or bed elevation; and the effect of flow constrictions such as dike fields.

98. It was determined that operation of the Bonnet Carre Spillway during a six-week flood of 1,400,000 cfs would result in deposition in the river downstream from the spillway. It would take several years for this flood deposit to move into Southwest Pass where it would influence dredging requirements. If Bonnet Carre Spillway was not operated this extra material would be delivered to Southwest Pass during the flood, while the extra discharge would be available to transport some of it to the Gulf.

99. The numerical model study determined that the effect of diverting water and sediment at Bonnet Carre Spillway will be to increase deposition downstream. This occurs because the reduced discharge will not be able to maintain the existing sediment concentration. However, this effect will not immediately be noticed in the lower reaches because the excess sediment will deposit upstream and will take several years to work its way down to Southwest

Pass. After about 12 years, dredging requirements in Southwest Pass would increase.

100. The stability of a proposed sediment sill to inhibit saltwater intrusion during extremely low flow conditions was evaluated. The sill was determined to be stable when the discharge was less than 400,000 cfs. Sill erosion rates increase with discharge above 400,000 cfs; at 800,000 cfs the sill washes out in about five days. The sill has no significant effect on upstream water-surface elevations. Using hydrographic survey data from the sill constructed in 1988, the model was found to reproduce the general decline in bed elevation as the discharge increased. It did not reproduce the slow downstream progression of the sill as it washed out.

101. The effect of diverting 10 percent of the Mississippi River at river mile 6.7 on dredging and deposition in Southwest Pass was tested with the numerical model. The diversion would increase the total of annual dredging and deposition between 13 and 22 percent depending on the sand concentration in the diversion.

102. The effect of diverting 70 percent of the Mississippi River upstream from Venice on dredging and deposition downstream was tested with the numerical model. Study results indicate that the net effect of a diversion would be an overall increase in dredging and deposition in the lower Mississippi River. However, in Southwest Pass, dredging and deposition would be decreased. For all diversion locations tested, the overall increase in lower Mississippi River net dredging and deposition was less with all secondary passes and outlets closed than with the existing Supplement II conditions. The results also indicate that significant differences in dredging and deposition occur depending on the assumed sand concentration in the diversion. The location of the diversion had a less significant effect on dredging and deposition than did closing the outlets. Because the river would tend to narrow and fill, the diversion schemes would create significant changes to the river regime downstream. Upstream the water-surface elevations would be lowered, resulting in channel degradation and increased sediment supply to downstream reaches during the 32-year simulation period.

103. The numerical model was used for preliminary evaluation of dike field designs for Redeye Crossing at river mile 224. Reliability of the one-dimensional numerical model was tested by comparing calculated annual dredging quantities with reported annual dredging quantities at Redeye Crossing for the

period between 1975 and 1989. The sum of calculated dredging volumes over a 15-year period was within 5 percent of the sum of reported dredging volumes. The dike plans tested in the numerical model indicated significant reduction in dredging requirements with the proposed dike fields.

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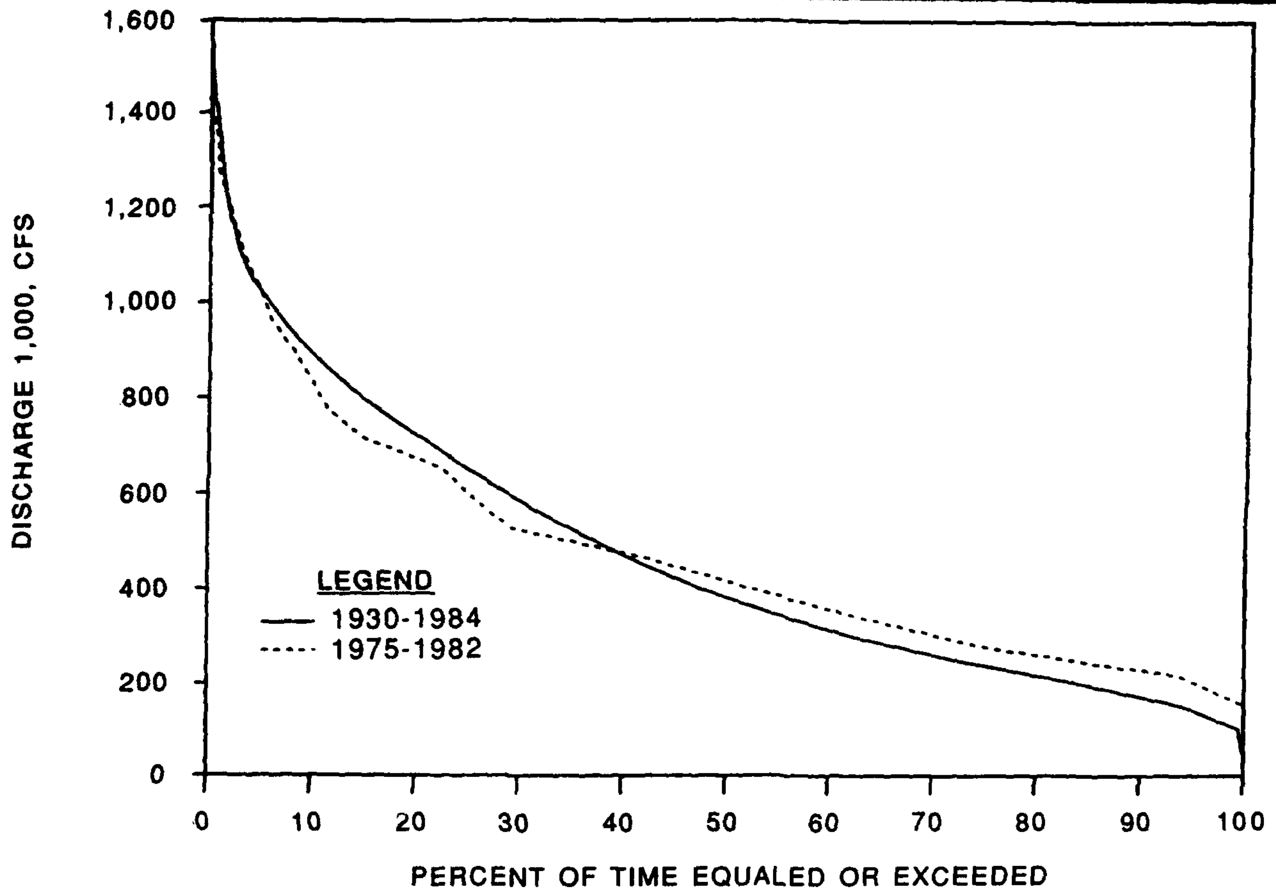


FLOW DURATION
MISSISSIPPI RIVER AT TARBERT LANDING

PLATE 1

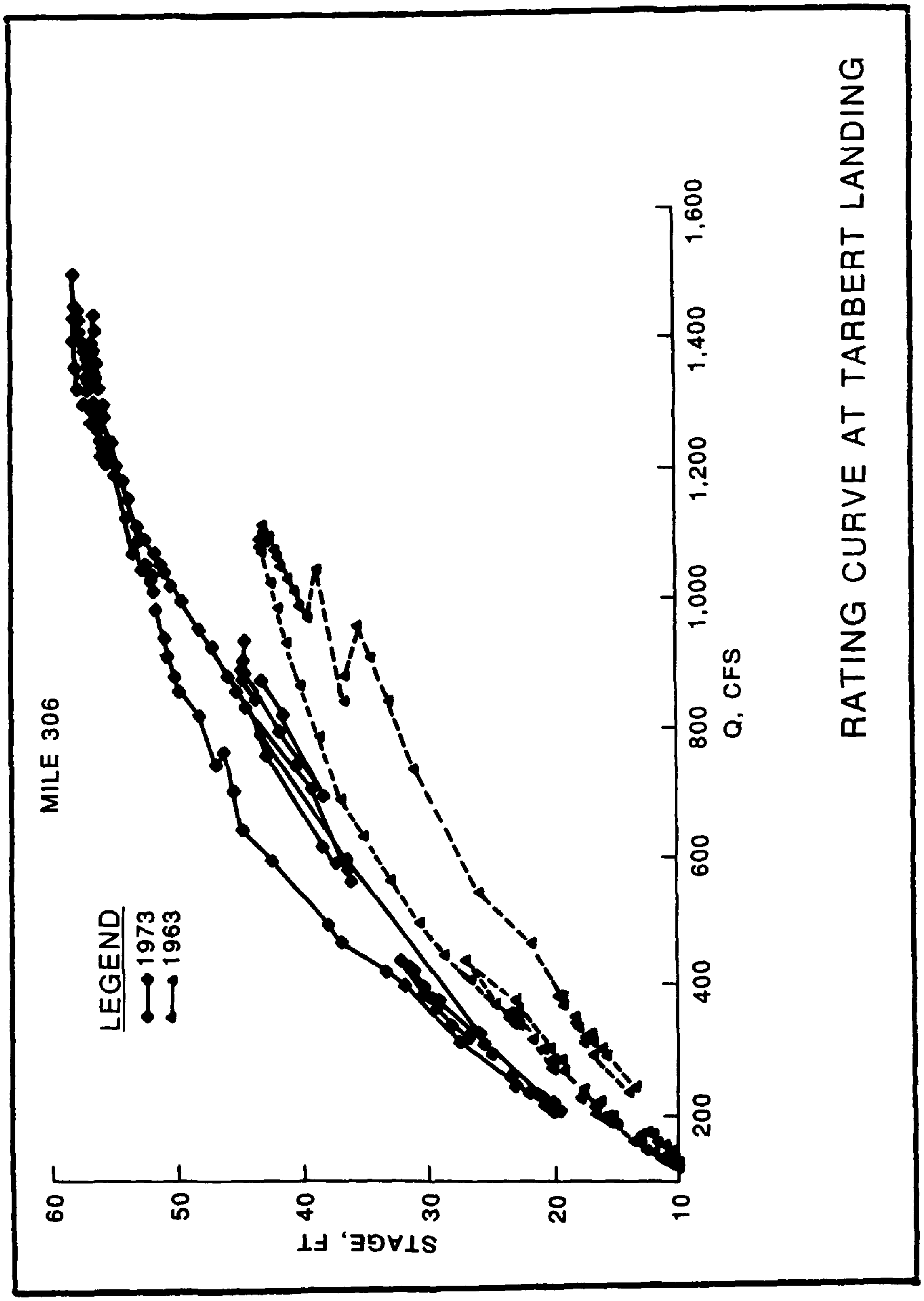
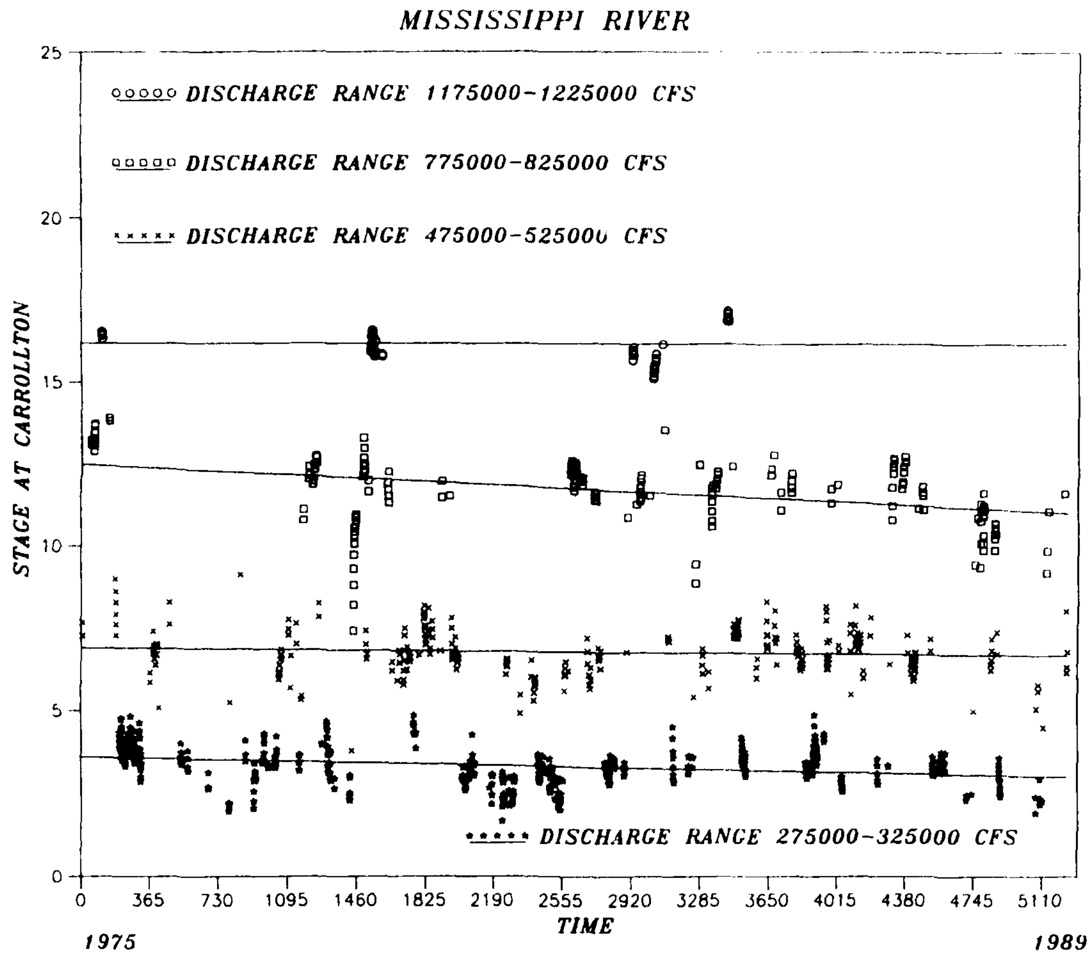
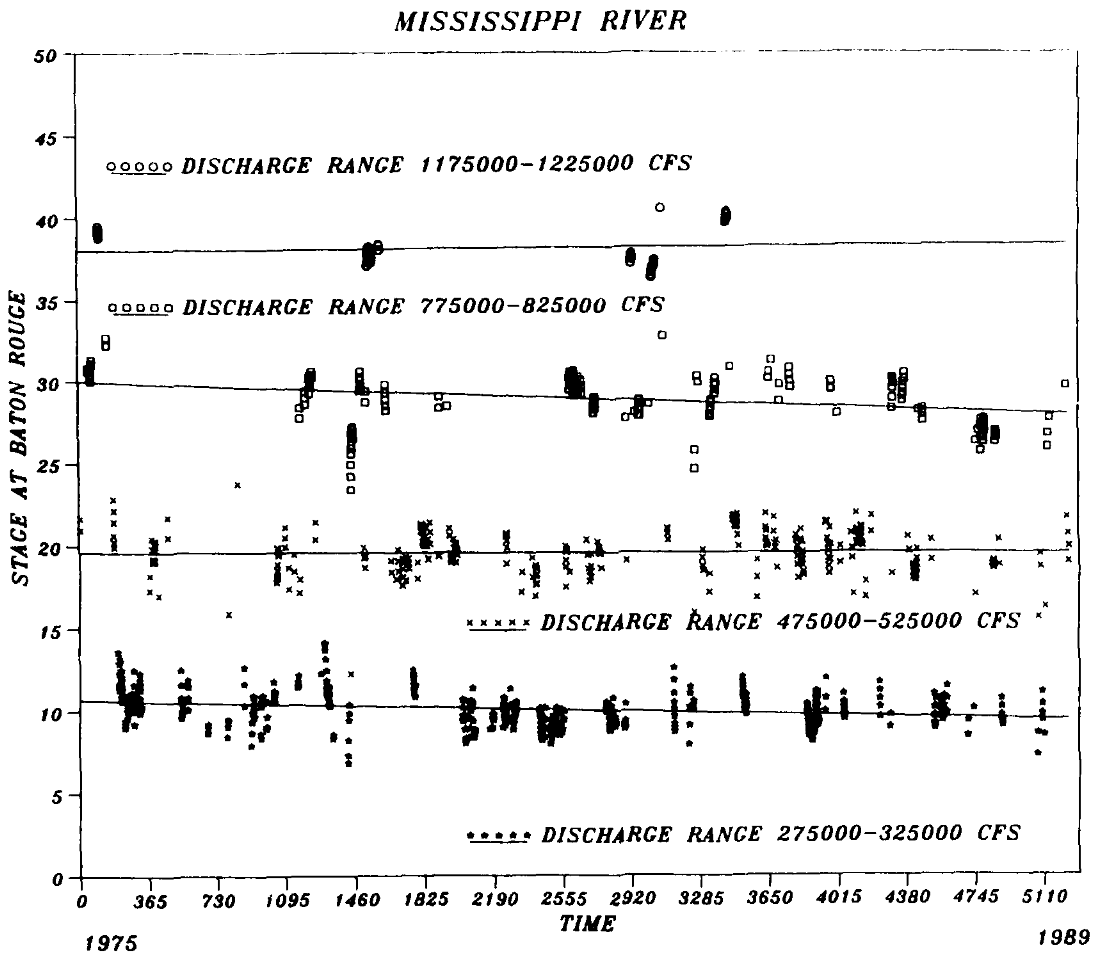

RATING CURVE AT TARBERT LANDING
MILE 306
LEGEND
1973
1963
STAGE, FT
60
50
40
30
20
10
Q, CFS
200
400
600
800
1,000
1,200
1,400
1,600

PLATE 2

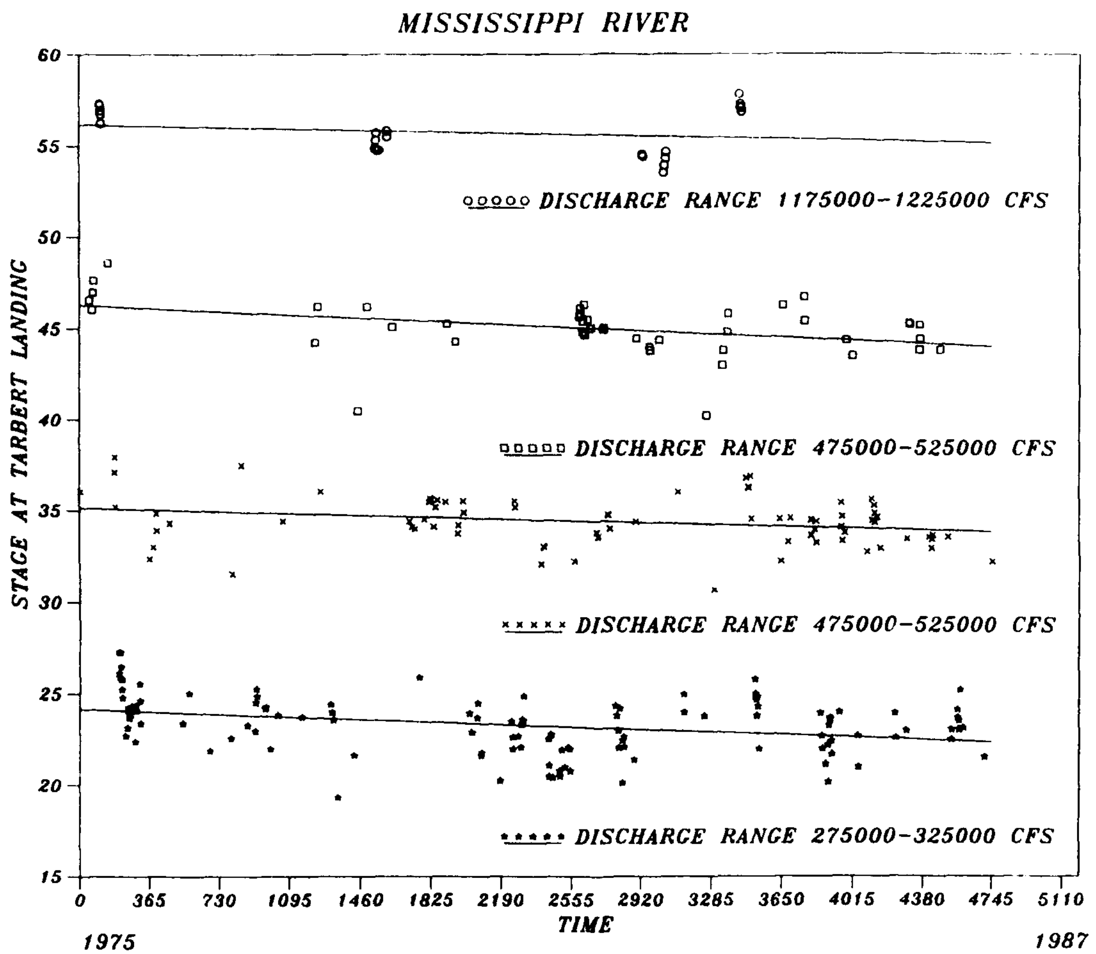


SPECIFIC GAGE CURVES
CARROLLTON



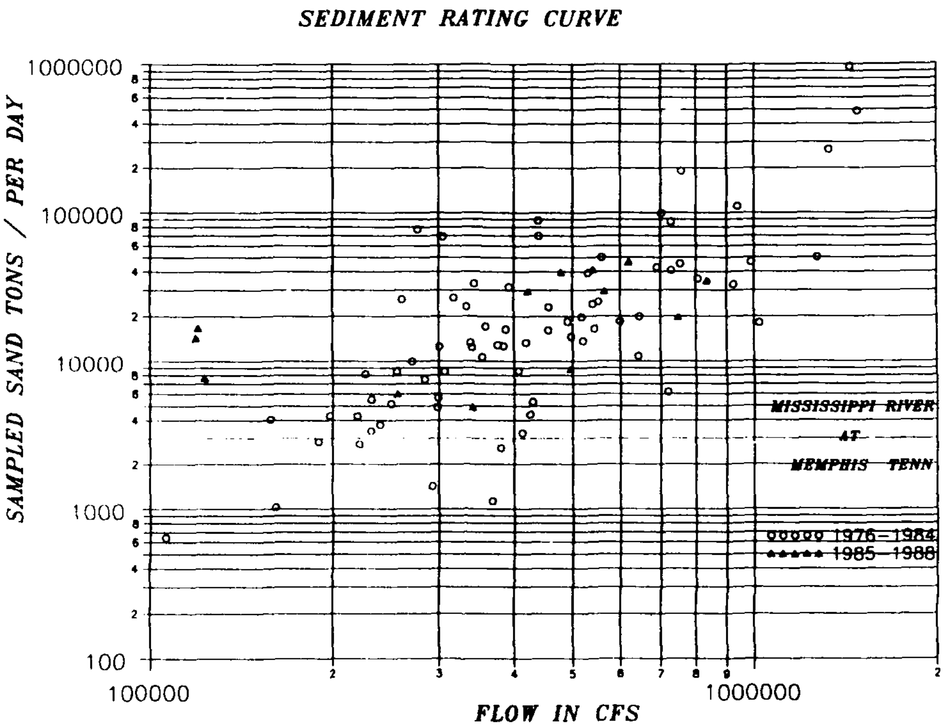
SPECIFIC GAGE CURVES
BATON ROUGE

PLATE 4



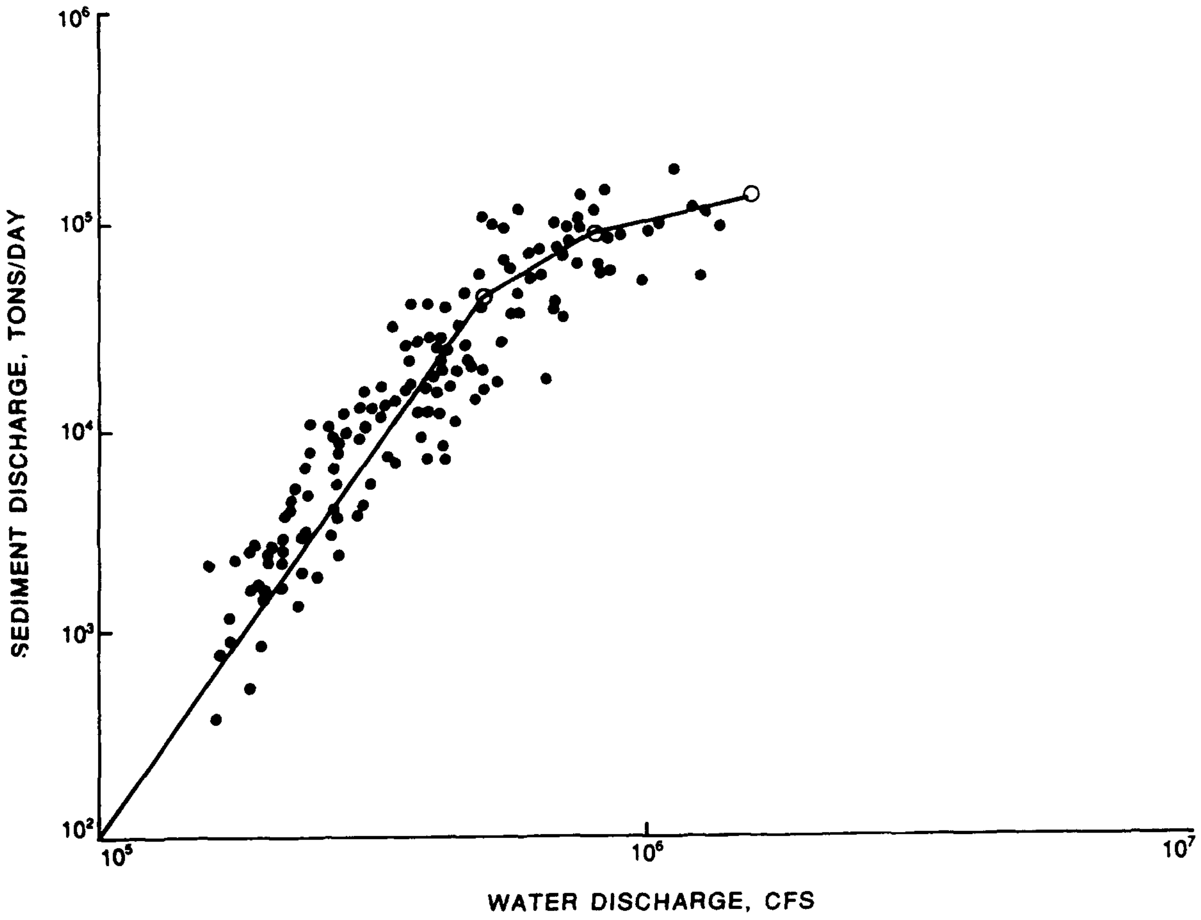
SPECIFIC GAGE CURVES
TARBERT LANDING

PLATE 5



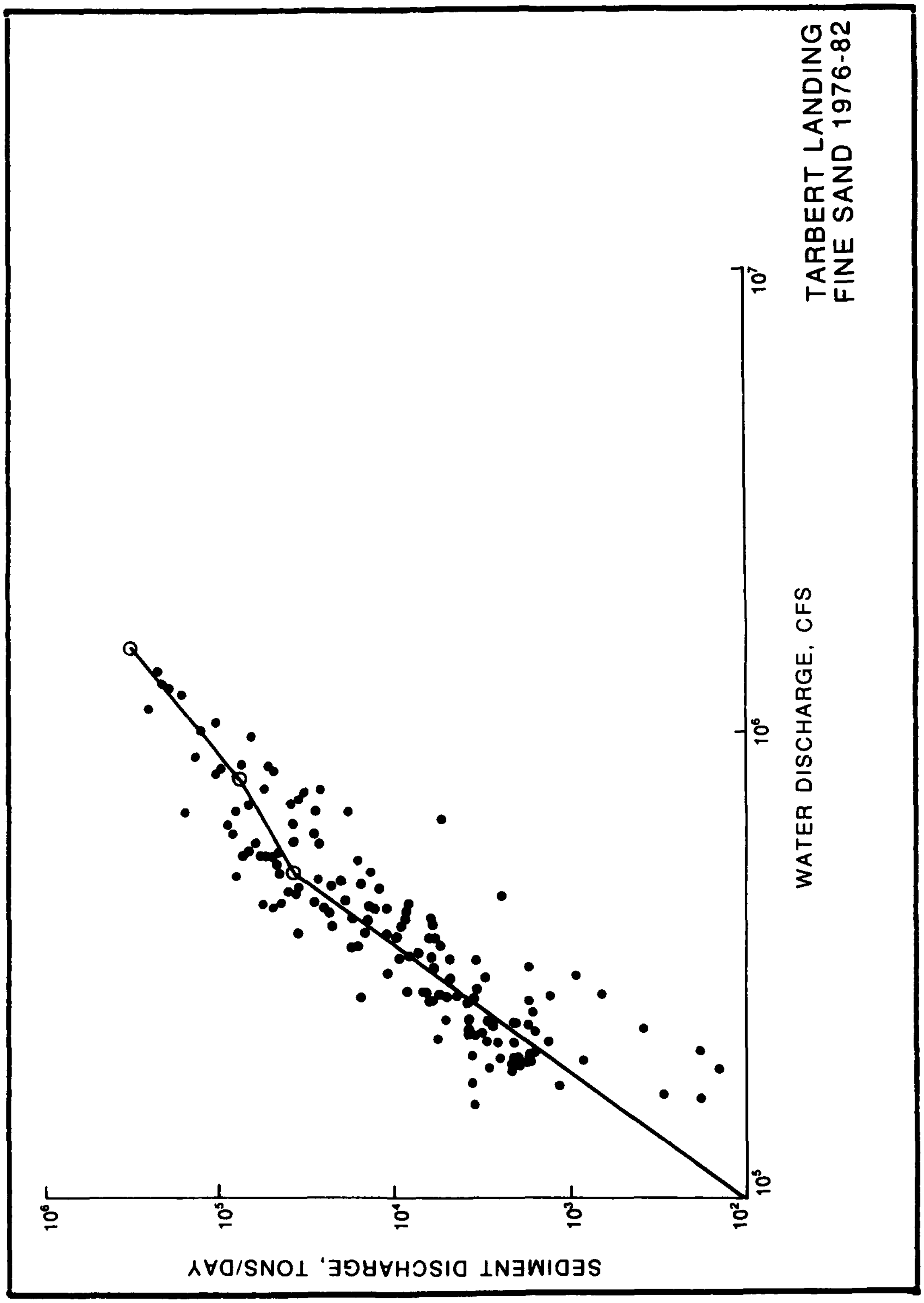
SAMPLED SAND LOAD AT MEMPHIS
1975-88

PLATE 6



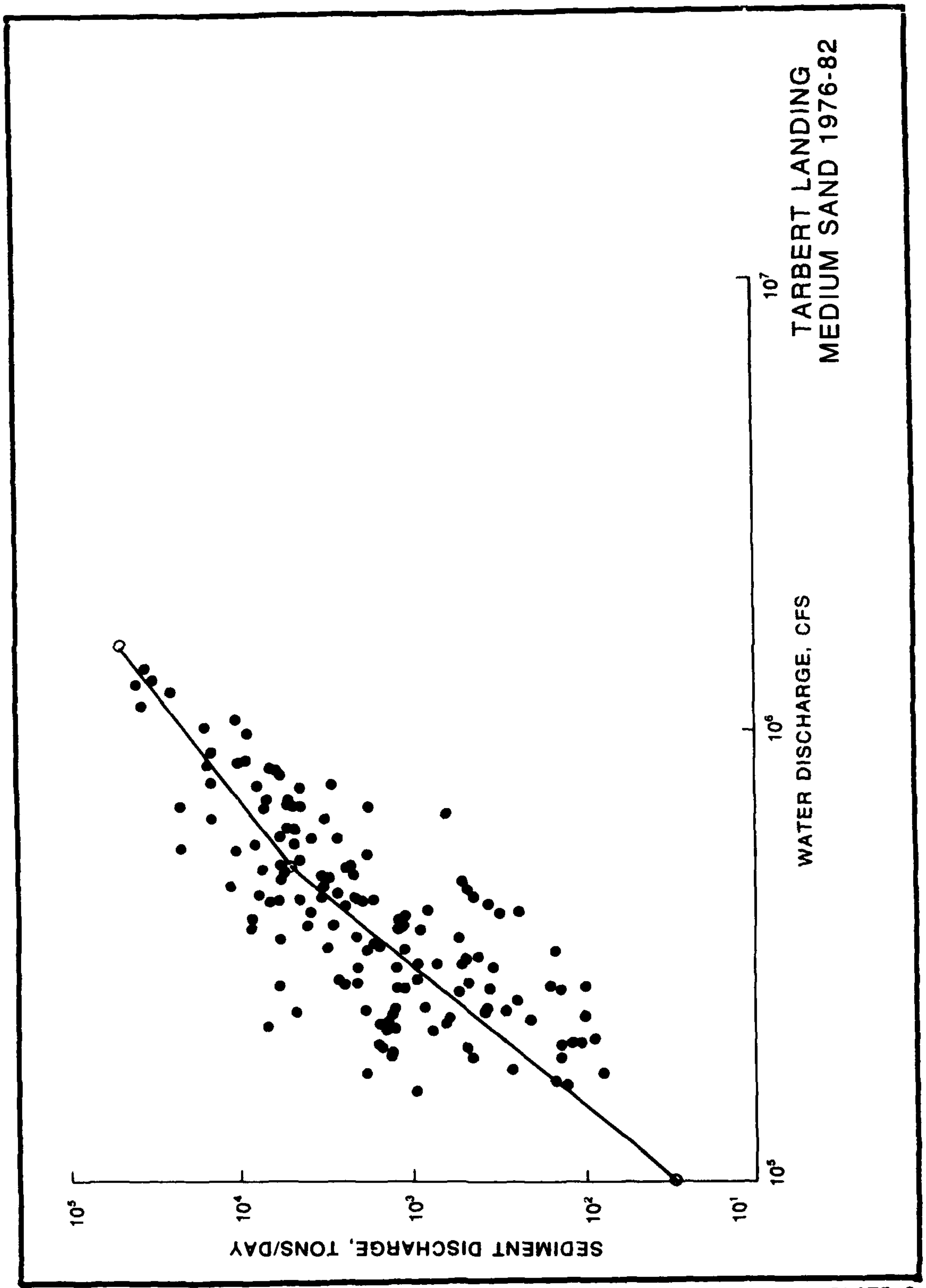
TARBERT LANDING
VERY FINE SAND 1976-82

PLATE 7



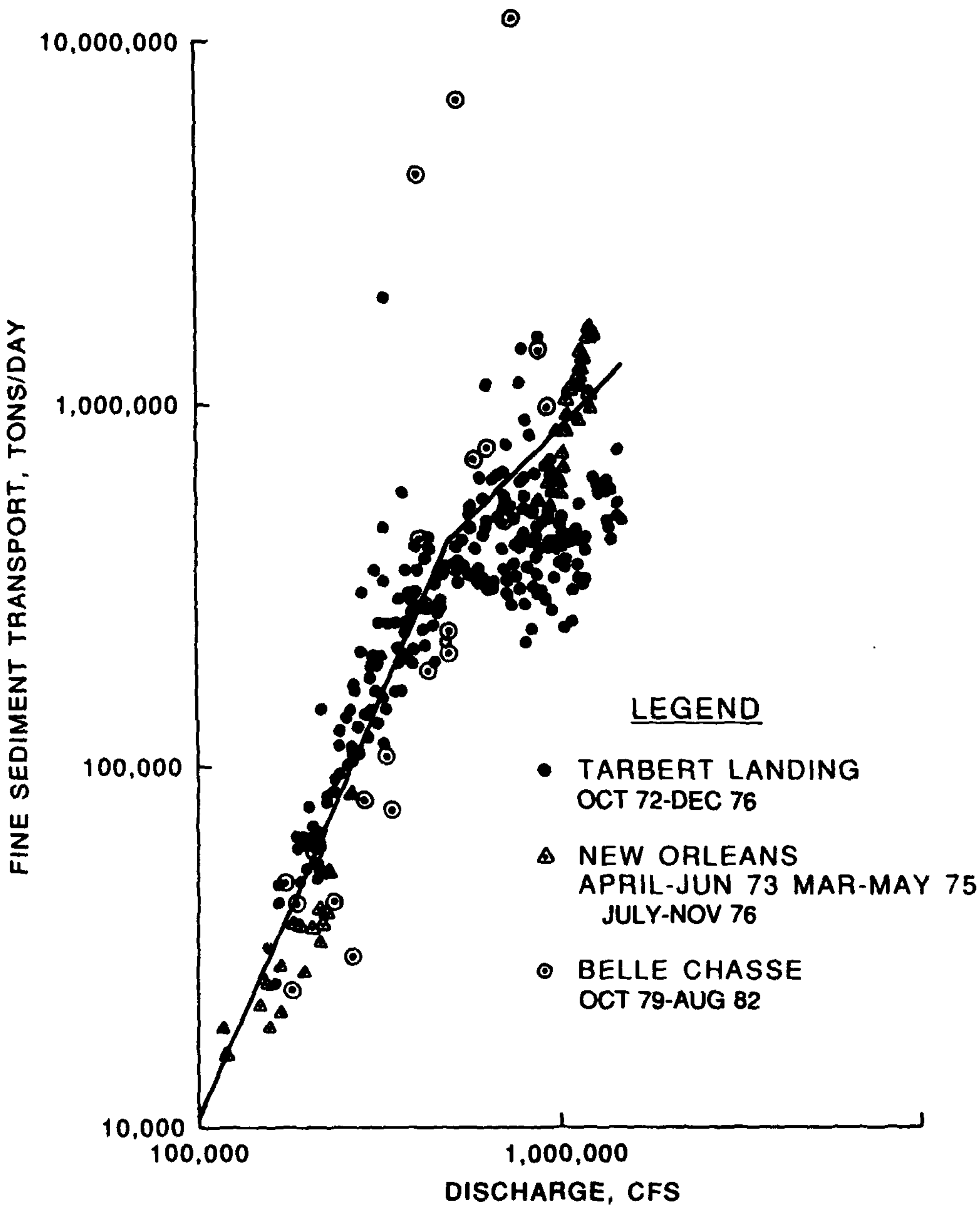
TARBERT LANDING
FINE SAND 1976-82
WATER DISCHARGE, CFS
10^5
10^6
10^7
SEDIMENT DISCHARGE, TONS/DAY
10^2
10^3
10^4
10^5
10^6

PLATE 8



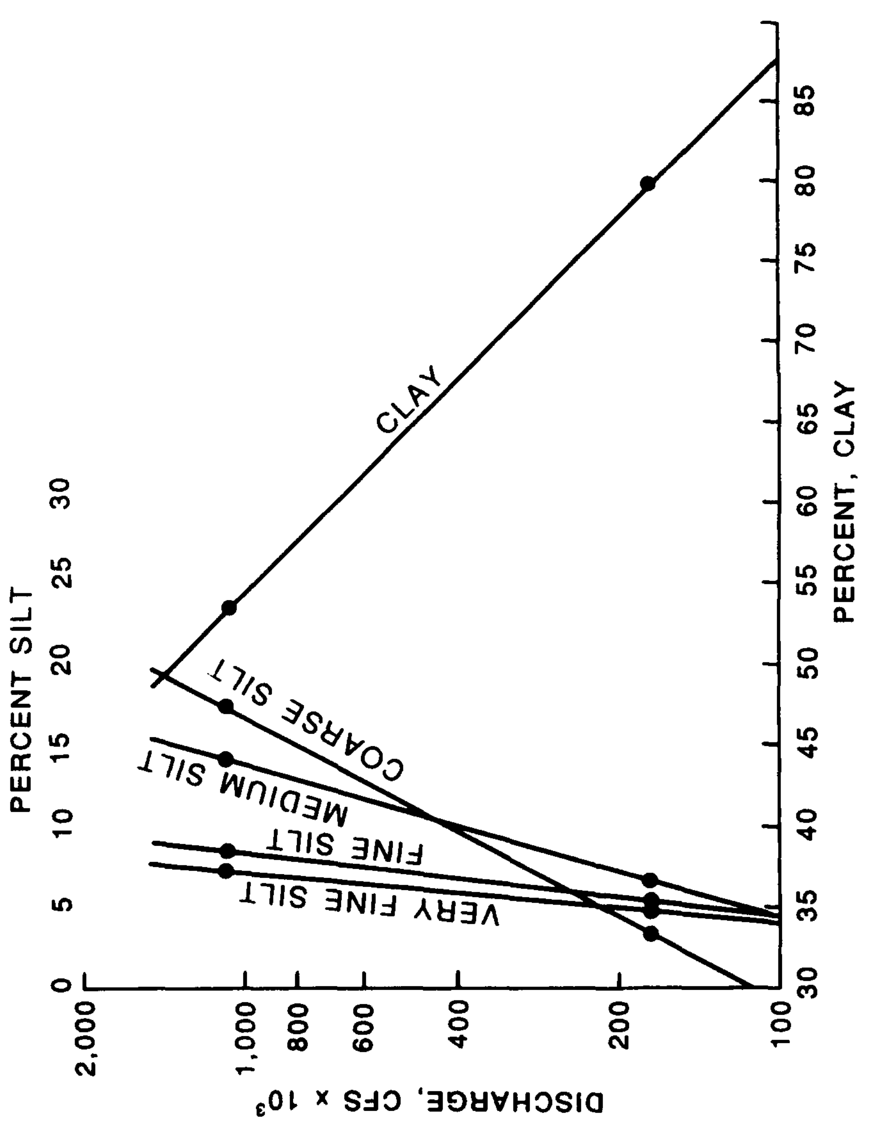
TARBERT LANDING
MEDIUM SAND 1976-82
WATER DISCHARGE, CFS
SEDIMENT DISCHARGE, TONS/DAY
10^5
10^6
10^7
10^1
10^2
10^3
10^4
10^5

PLATE 9

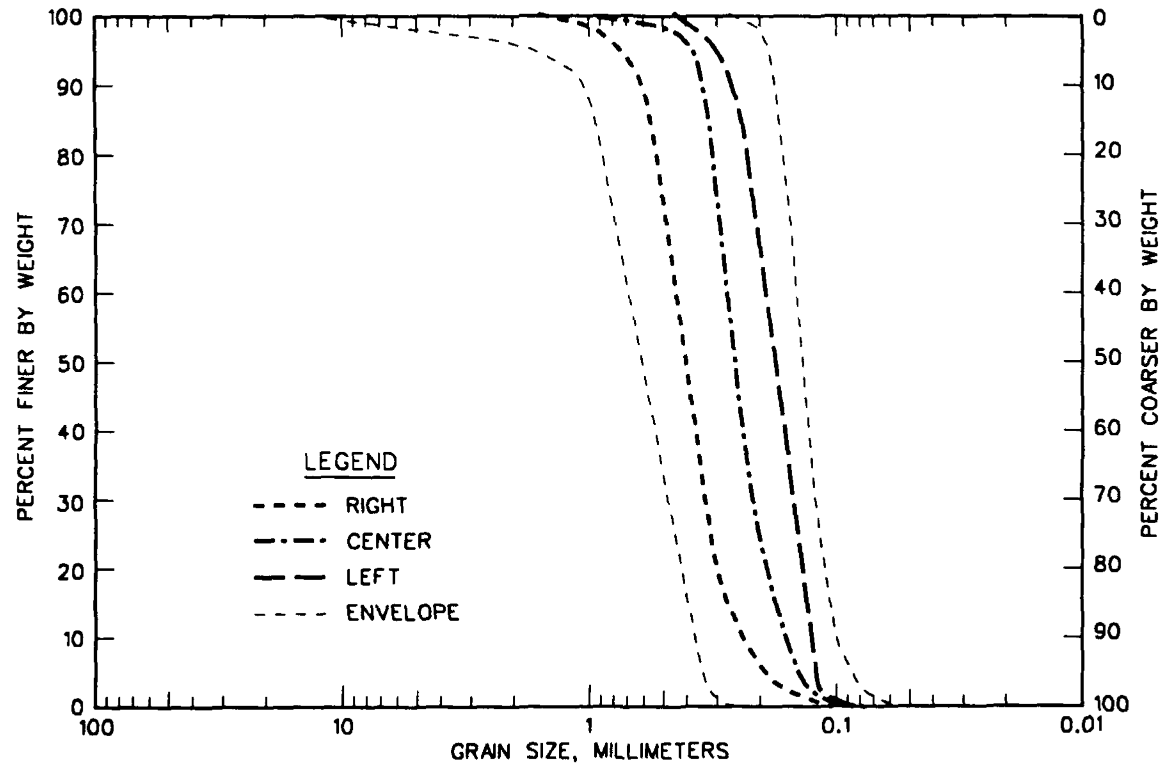


SEDIMENT DISCHARGE
SILTS & CLAYS
MISSISSIPPI RIVER

PLATE 10

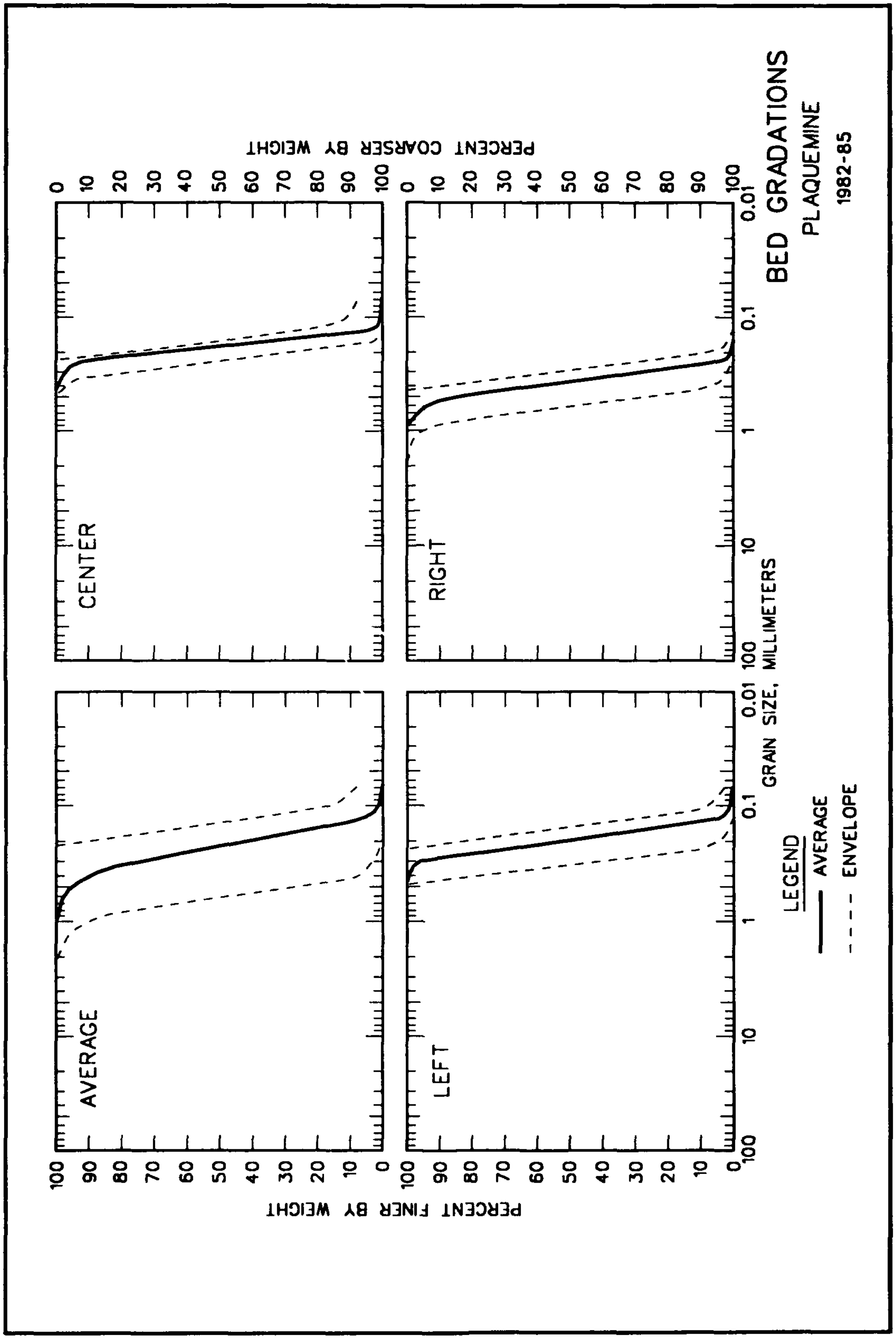


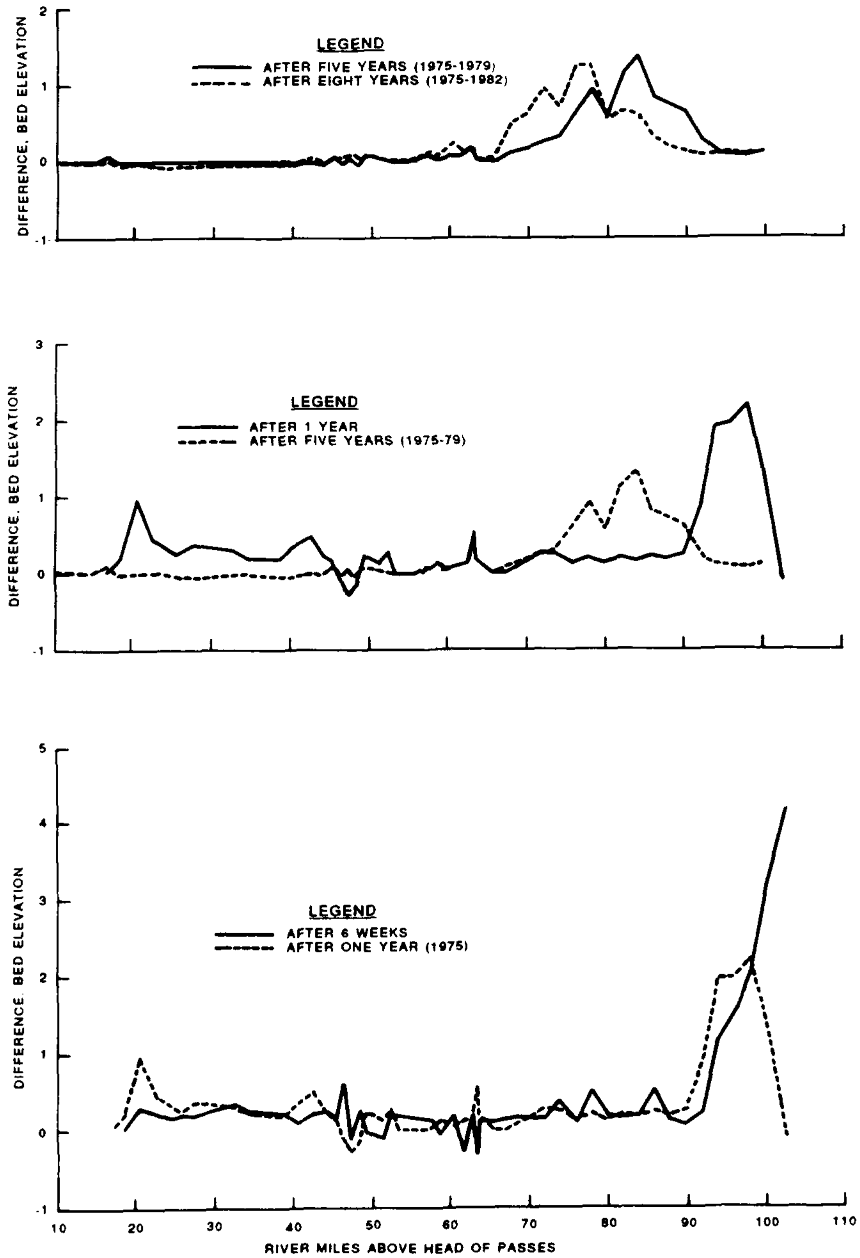
SEDIMENT DISTRIBUTION
OF FINES
NEW ORLEANS



BED GRADATIONS
ST. FRANCISVILLE
1982-85

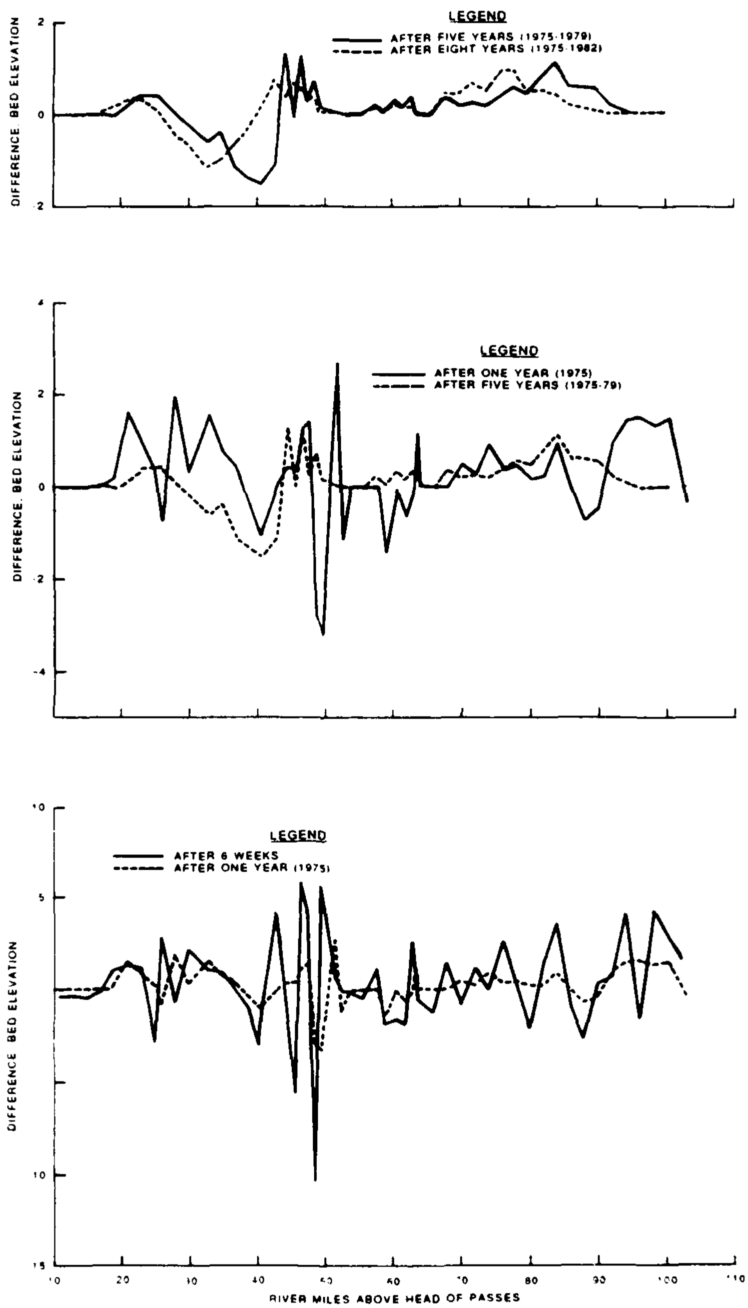
PLATE 12


BED GRADATIONS
PLAQUEMINE
1982-85
AVERAGE
CENTER
LEFT
RIGHT
PERCENT FINER BY WEIGHT
PERCENT COARSER BY WEIGHT
GRAIN SIZE, MILLIMETERS
100
10
1
0.1
0.01
0
10
20
30
40
50
60
70
80
90
100
LEGEND
AVERAGE
ENVELOPE

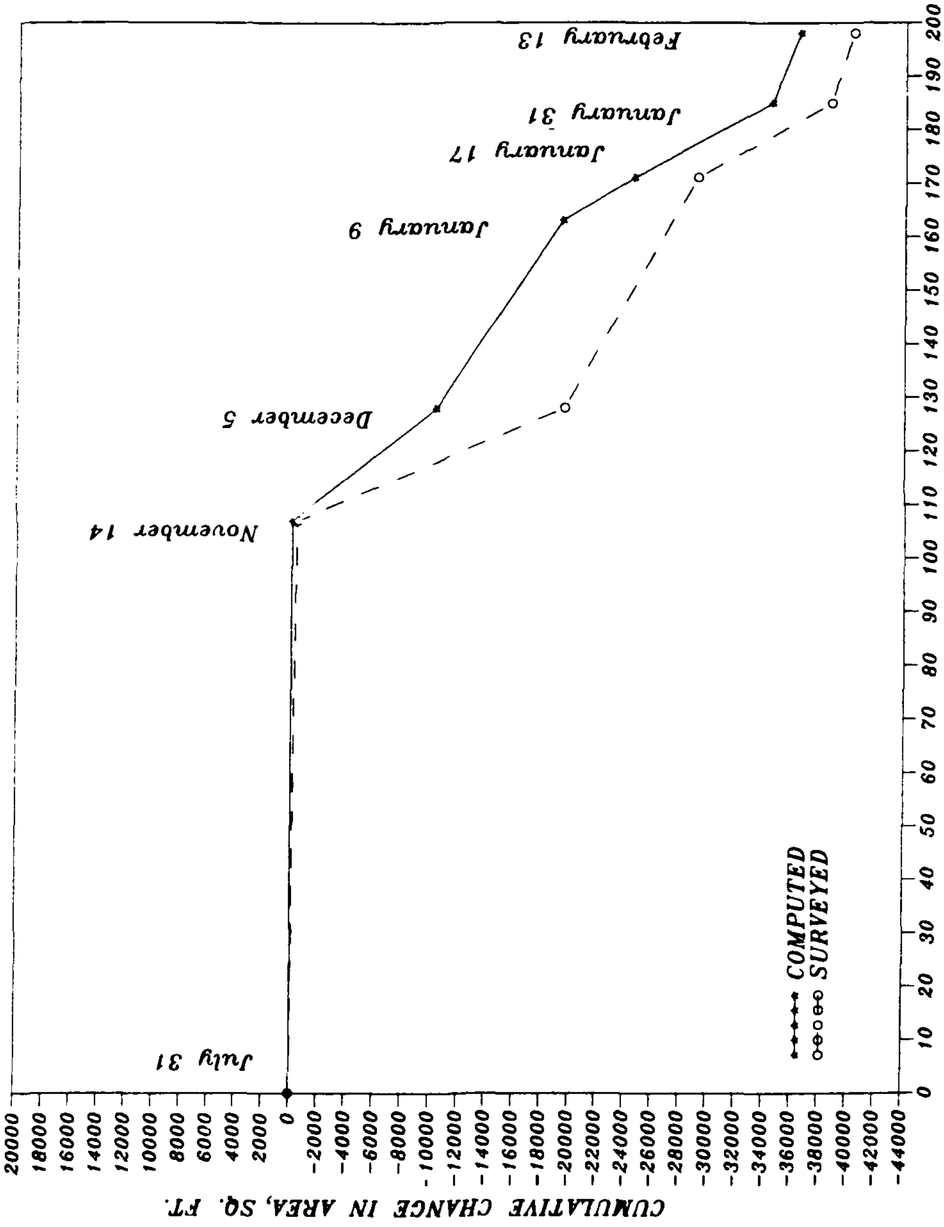


MISSISSIPPI RIVER VENICE TO CARROLLTON
BONNET CARRE OPEN
Q = 1,250,000 CFS

PLATE 14

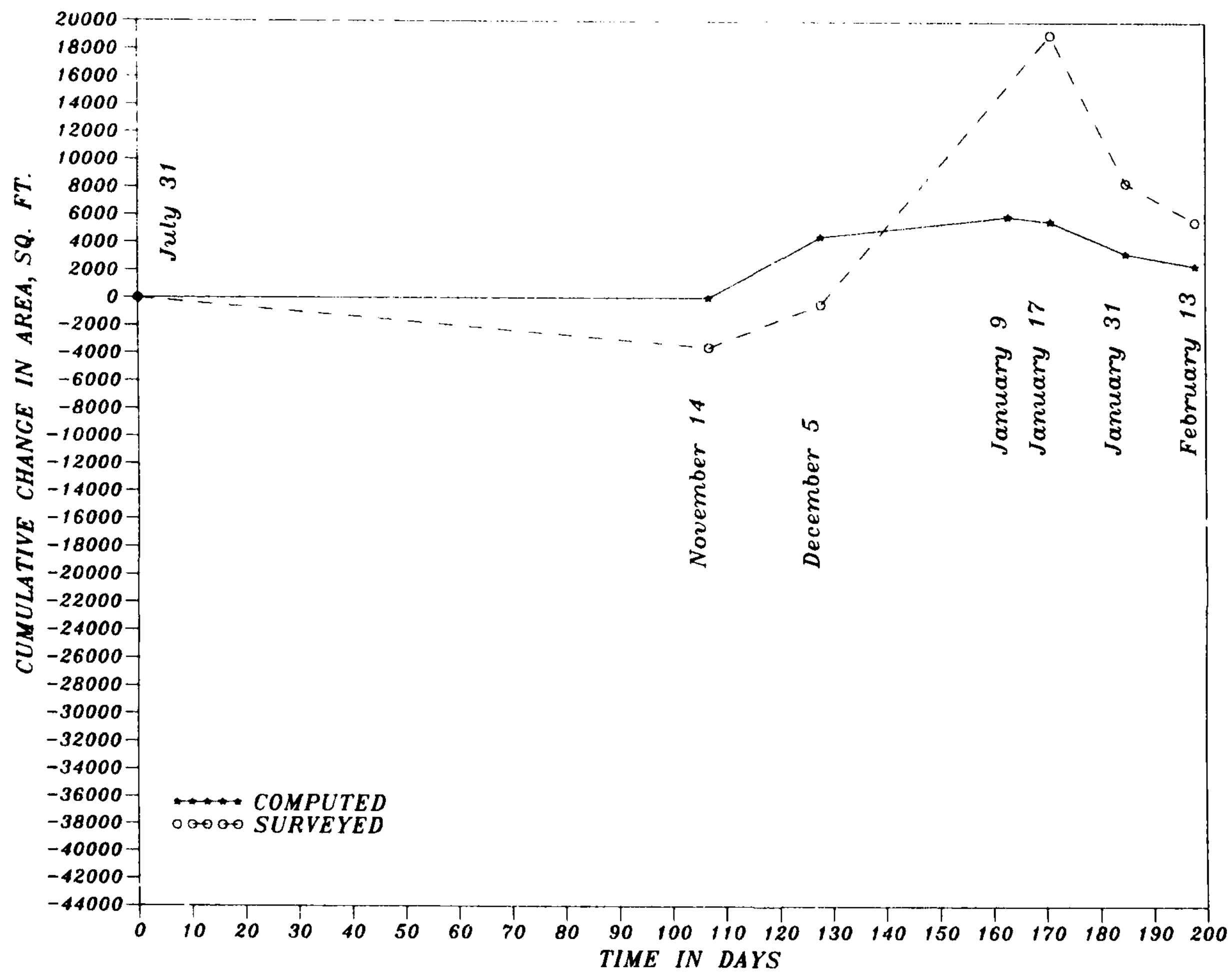


MISSISSIPPI RIVER, VENICE TO CARROLLTON
BONNET CARRE CLOSED
1,400,000



SURVEYED AND CALCULATED CUMULATIVE
CROSS-SECTIONAL AREA CHANGES AT MILE 63.4

PLATE 16



SURVEYED AND CALCULATED CUMULATIVE
CROSS-SECTIONAL AREA CHANGES 400 FT DOWNSTREAM
FROM MILE 63.4

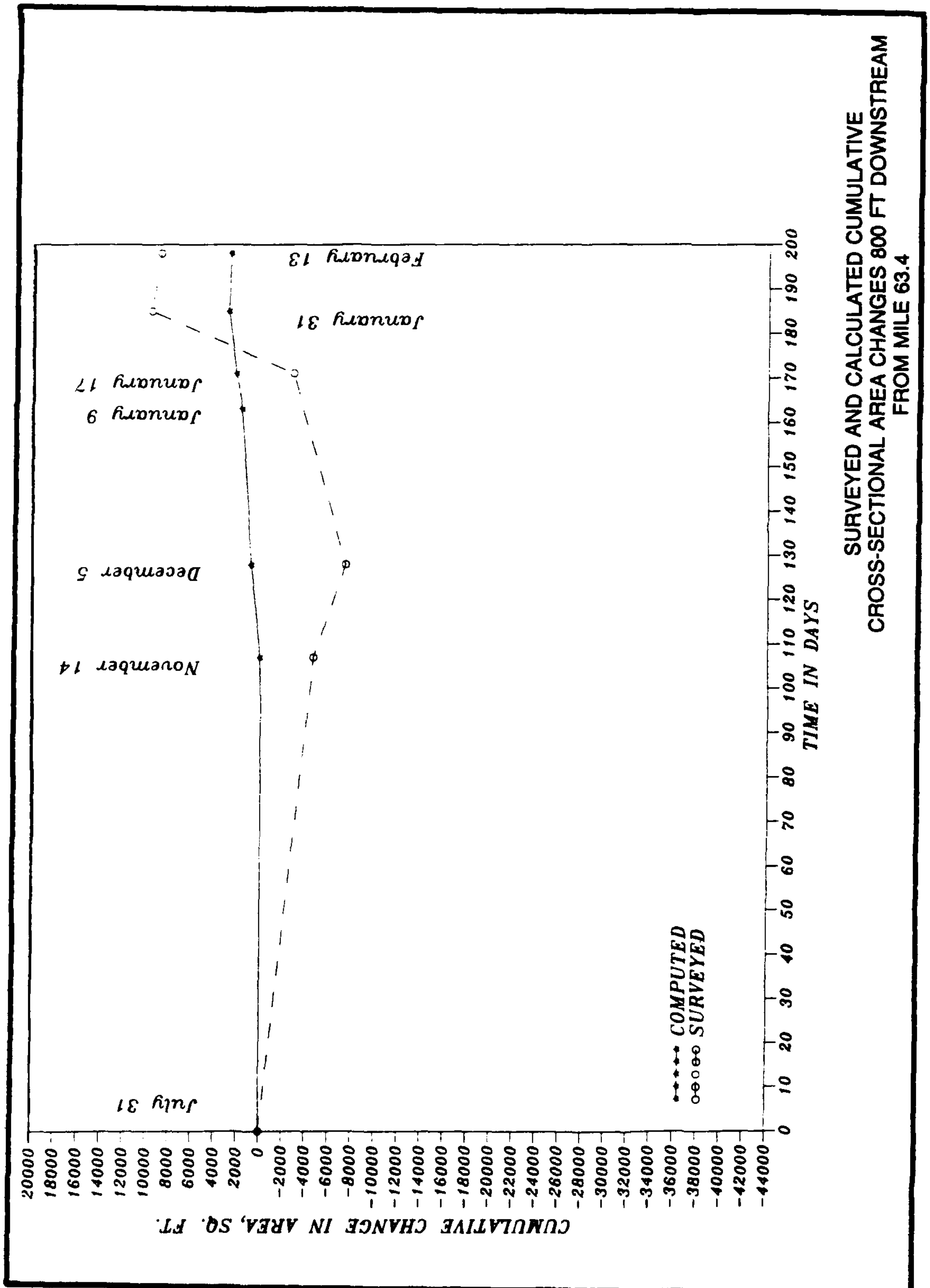
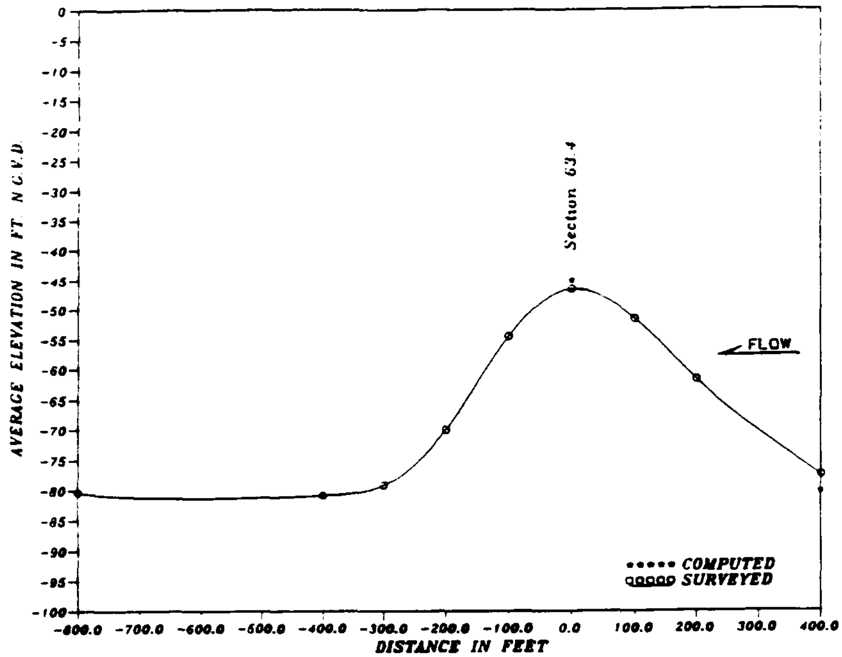
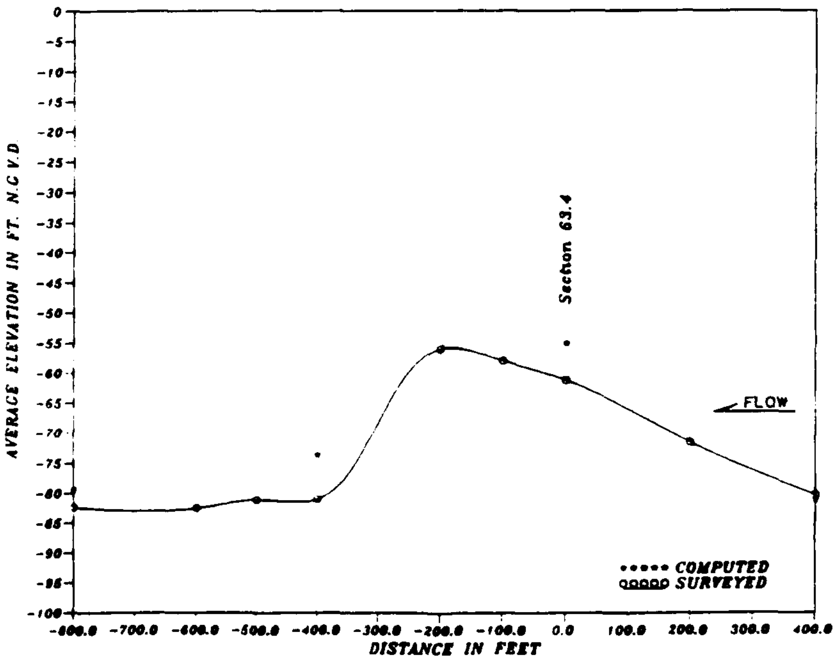

SURVEYED AND CALCULATED CUMULATIVE
CROSS-SECTIONAL AREA CHANGES 800 FT DOWNSTREAM
FROM MILE 63.4
TIME IN DAYS
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
CUMULATIVE CHANGE IN AREA, SQ. FT.
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
-10000
-12000
-14000
-16000
-18000
-20000
-22000
-24000
-26000
-28000
-30000
-32000
-34000
-36000
-38000
-40000
-42000
-44000
COMPUTED
SURVEYED
July 31
November 14
December 5
January 9
January 17
January 31
February 13

PLATE 18

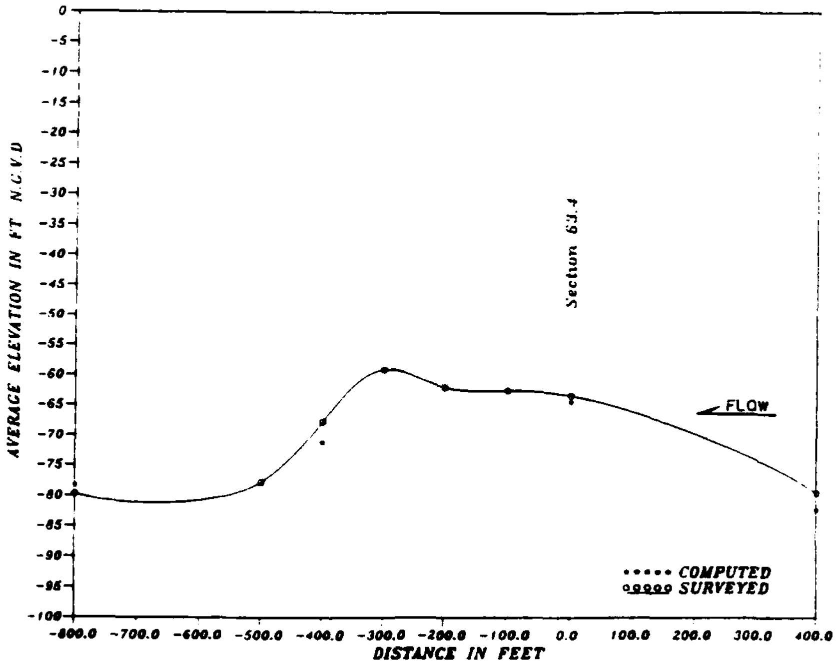


a. 14 November 1988

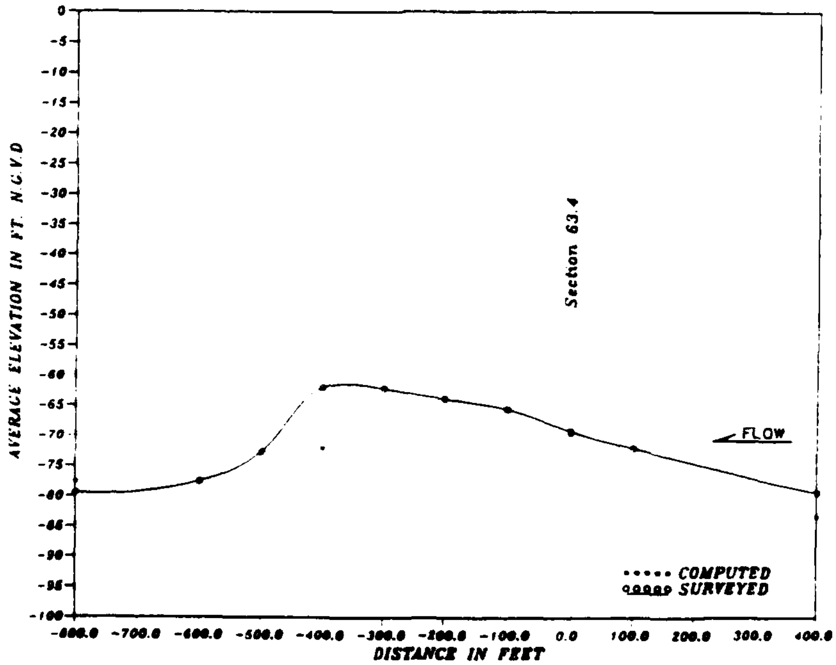


b. 5 December 1988

AVERAGE BED PROFILE
IN VICINITY OF SEDIMENT SILL



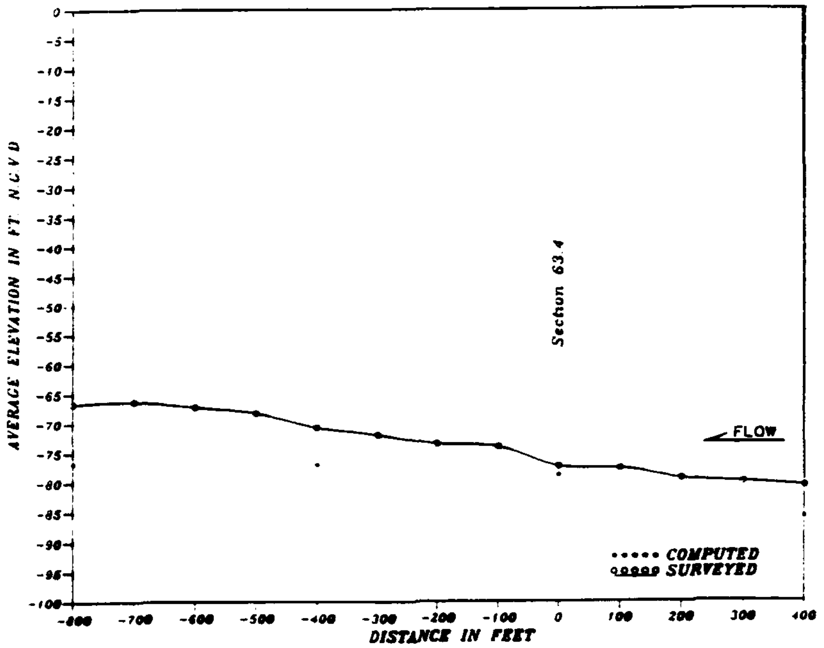
c. 9 January 1989



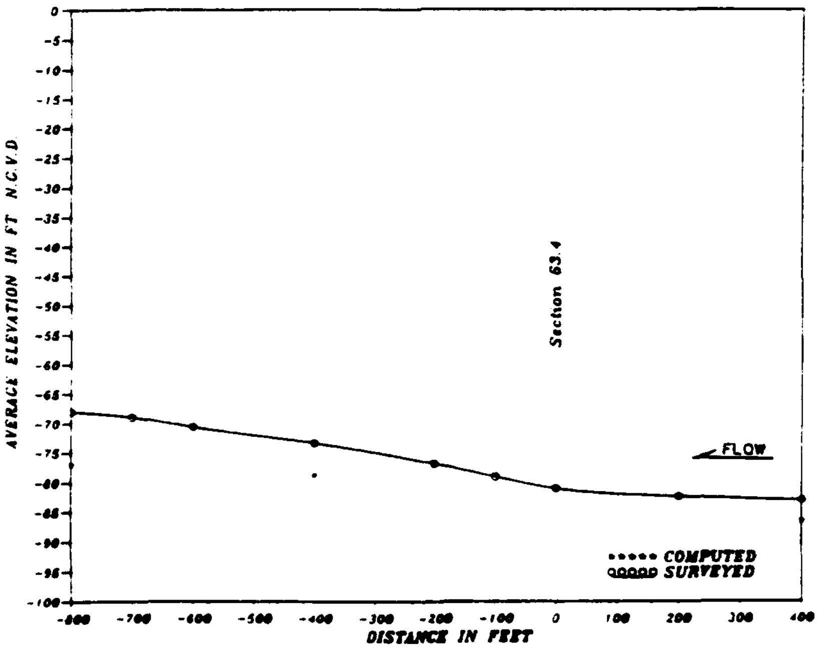
d. 17 January 1989

AVERAGE BED PROFILE
IN VICINITY OF SEDIMENT SILL

PLATE 19
(Sheet 2 of 3)

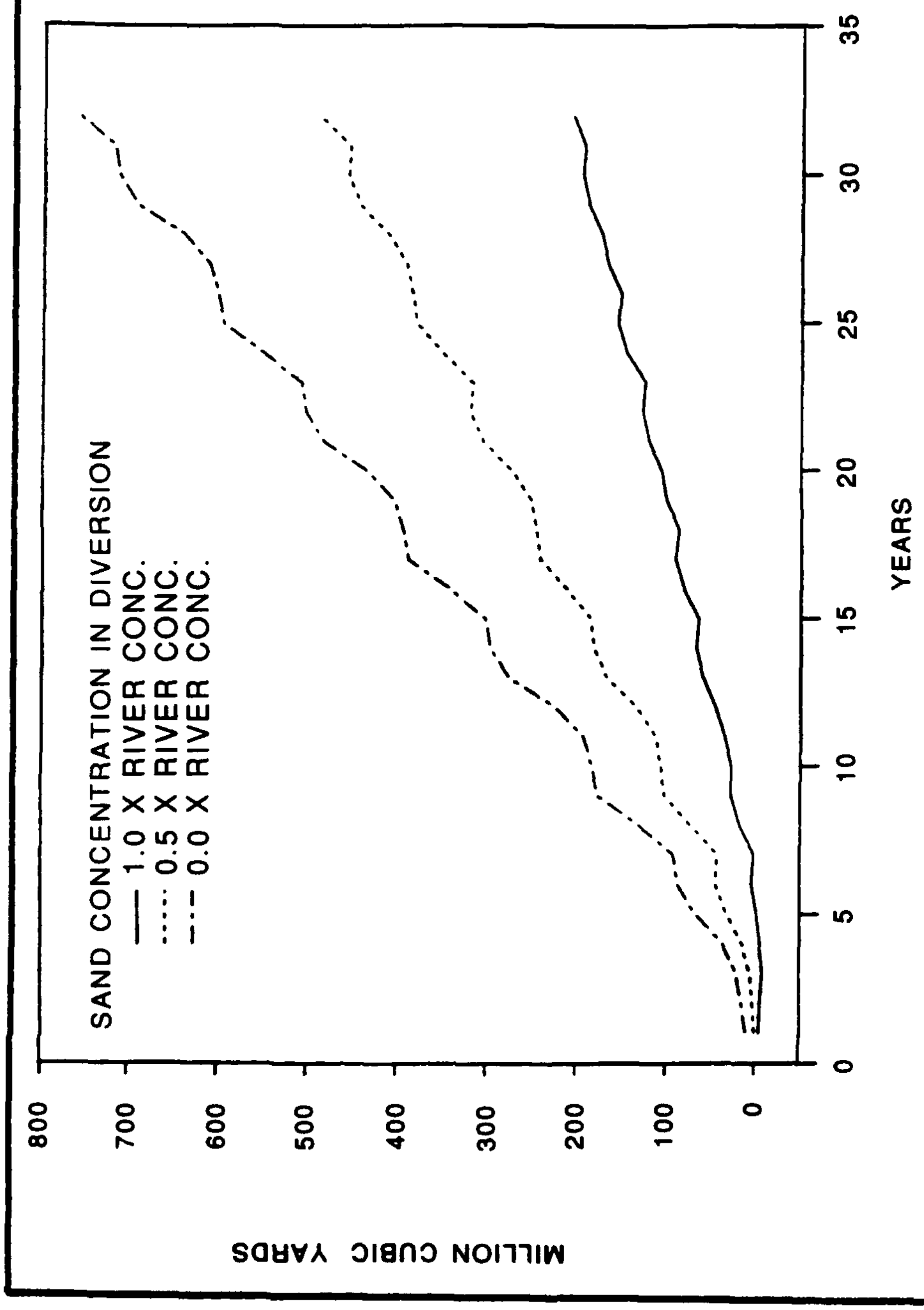


e. 31 January 1989



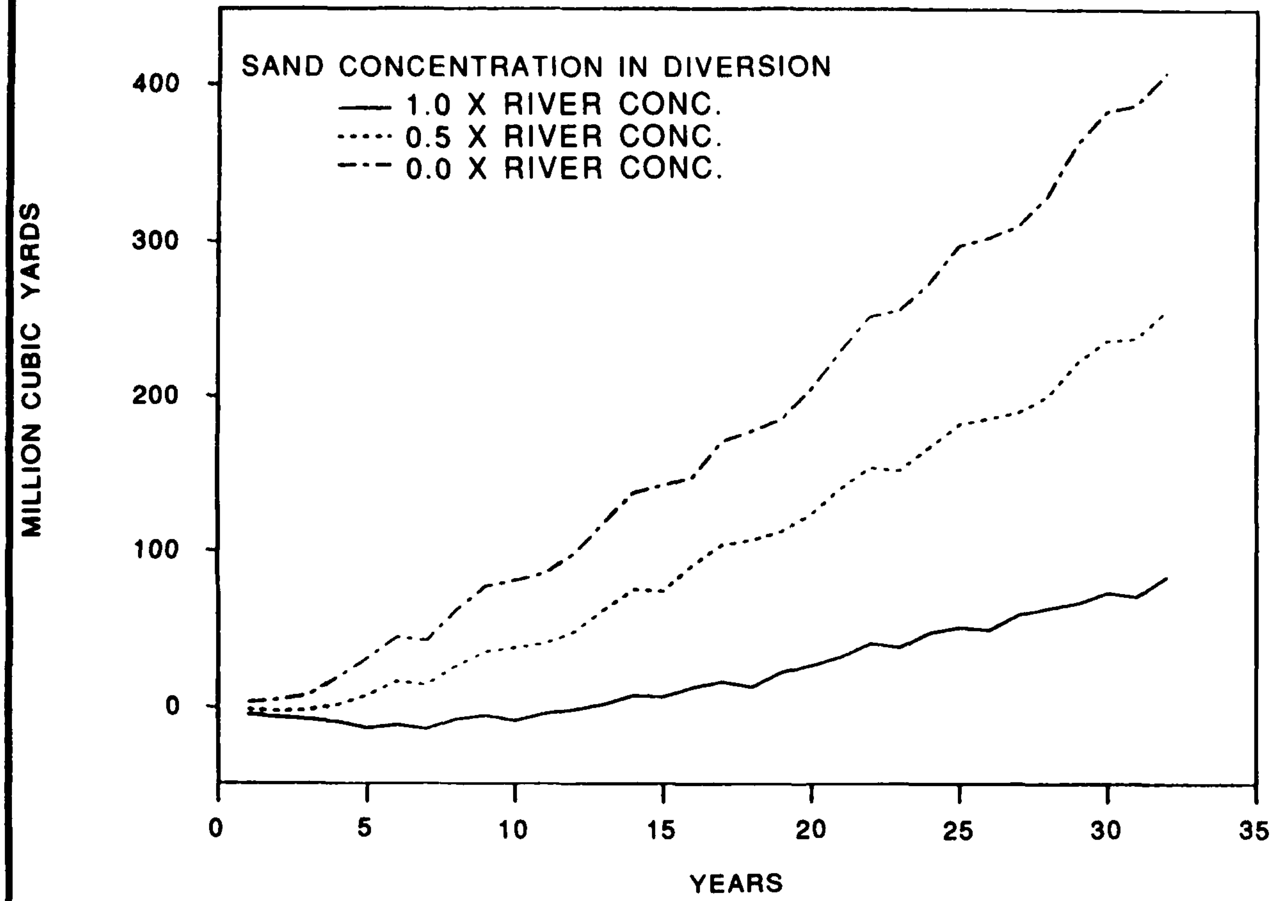
f. 13 February 1989

AVERAGE BED PROFILE
IN VICINITY OF SEDIMENT SILL

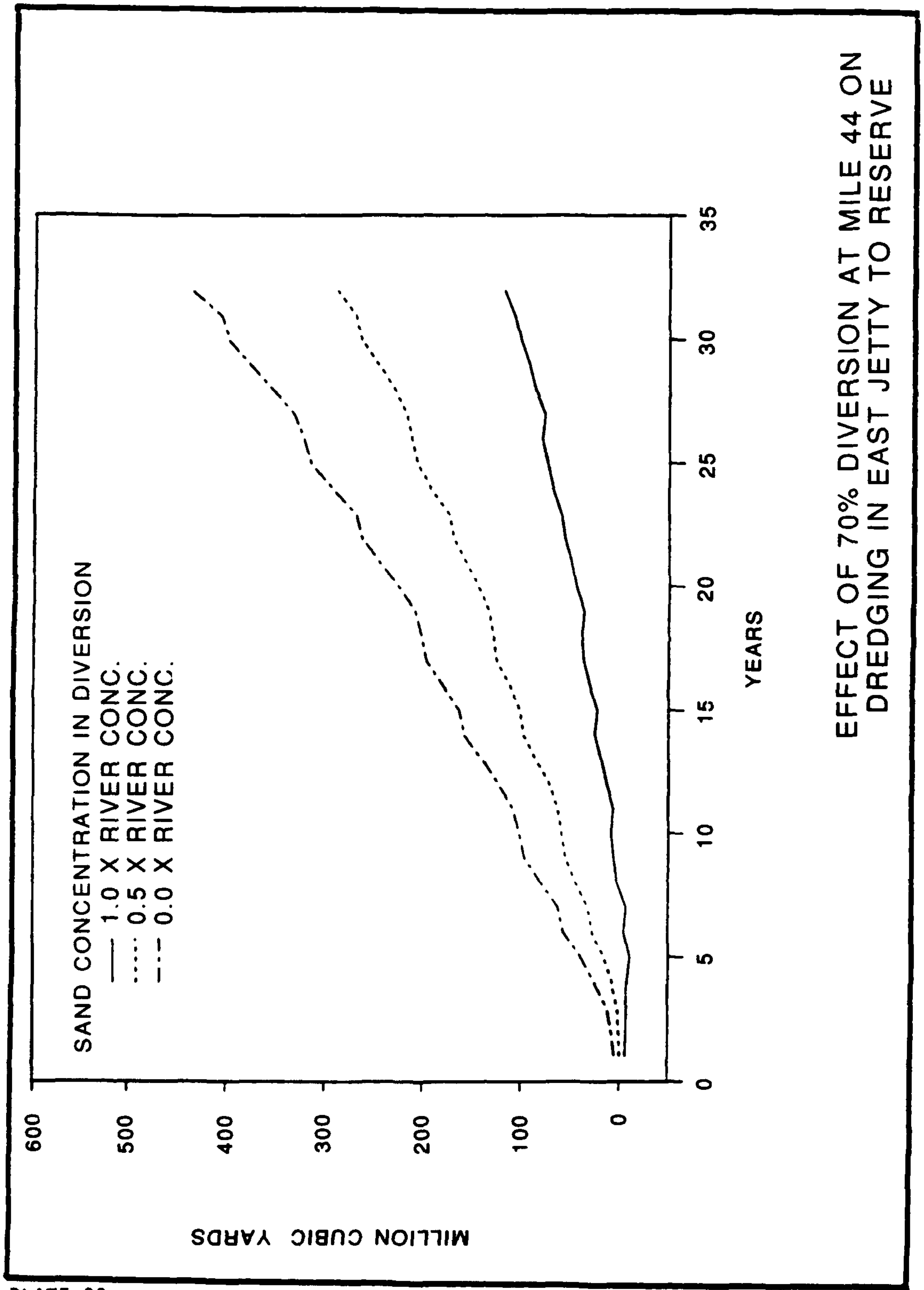


EFFECT OF 70% DIVERSION AT MILE 81
ON DREDGING IN EAST JETTY TO RESERVE

PLATE 20

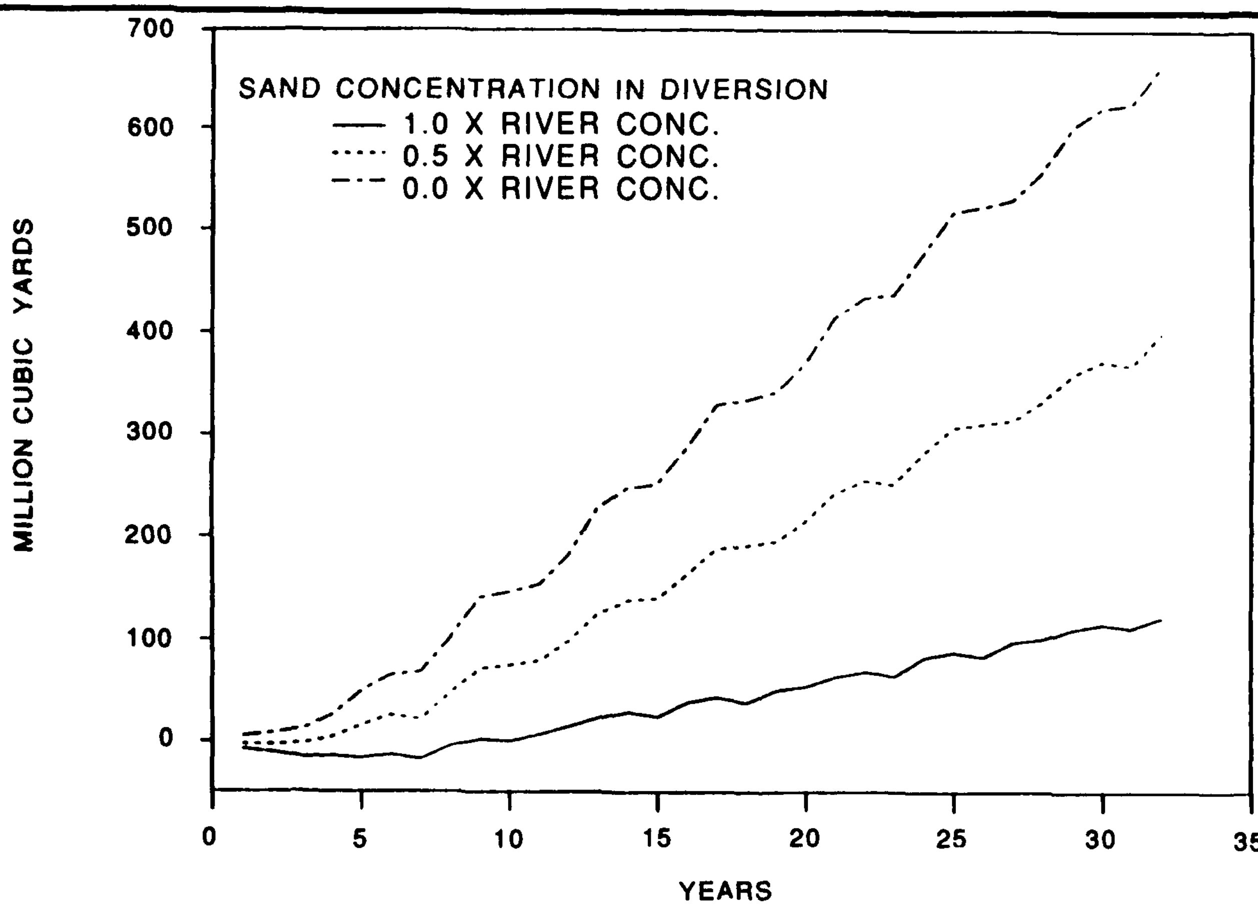


EFFECT OF 70% DIVERSION AT MILE 59
ON DREDGING IN EAST JETTY TO RESERVE

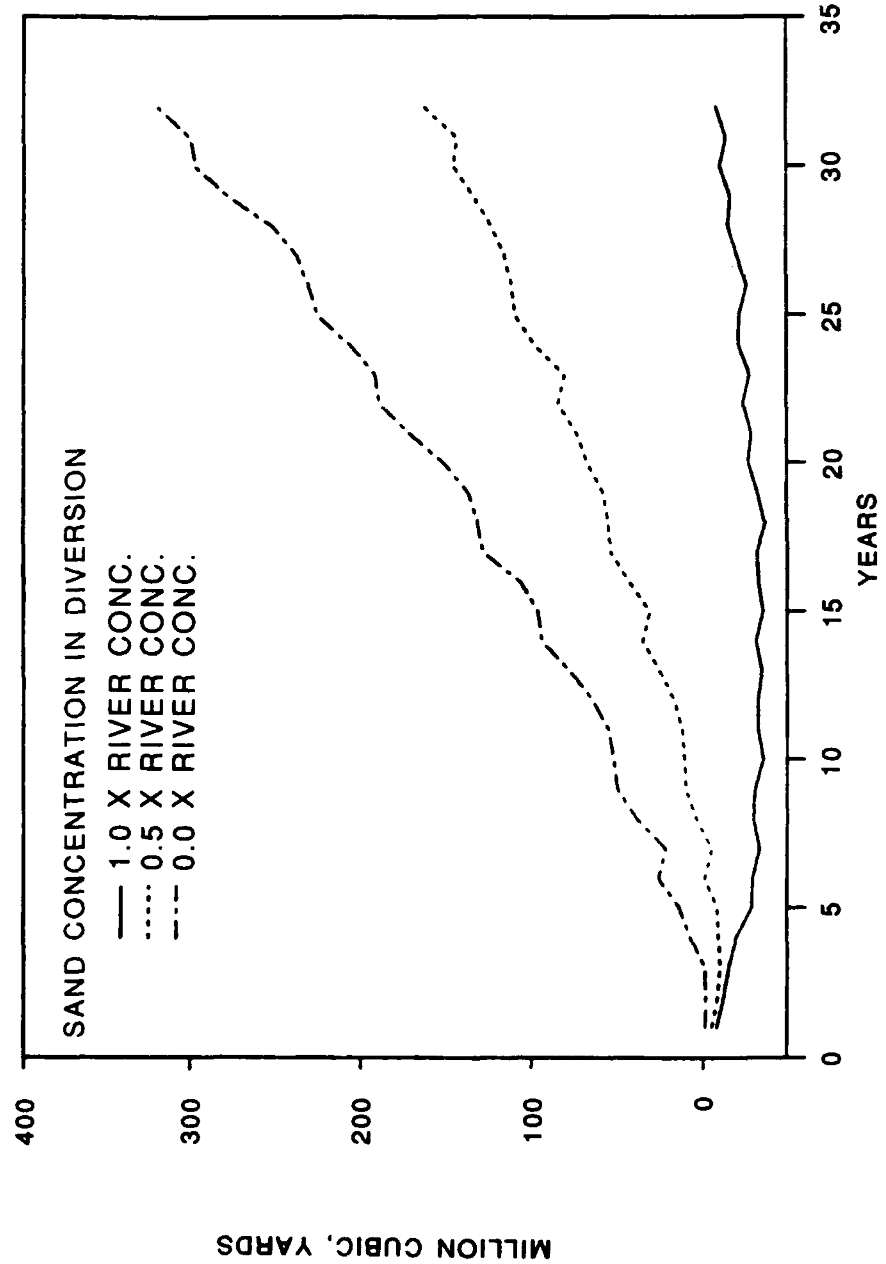


EFFECT OF 70% DIVERSION AT MILE 44 ON
DREDGING IN EAST JETTY TO RESERVE

PLATE 22

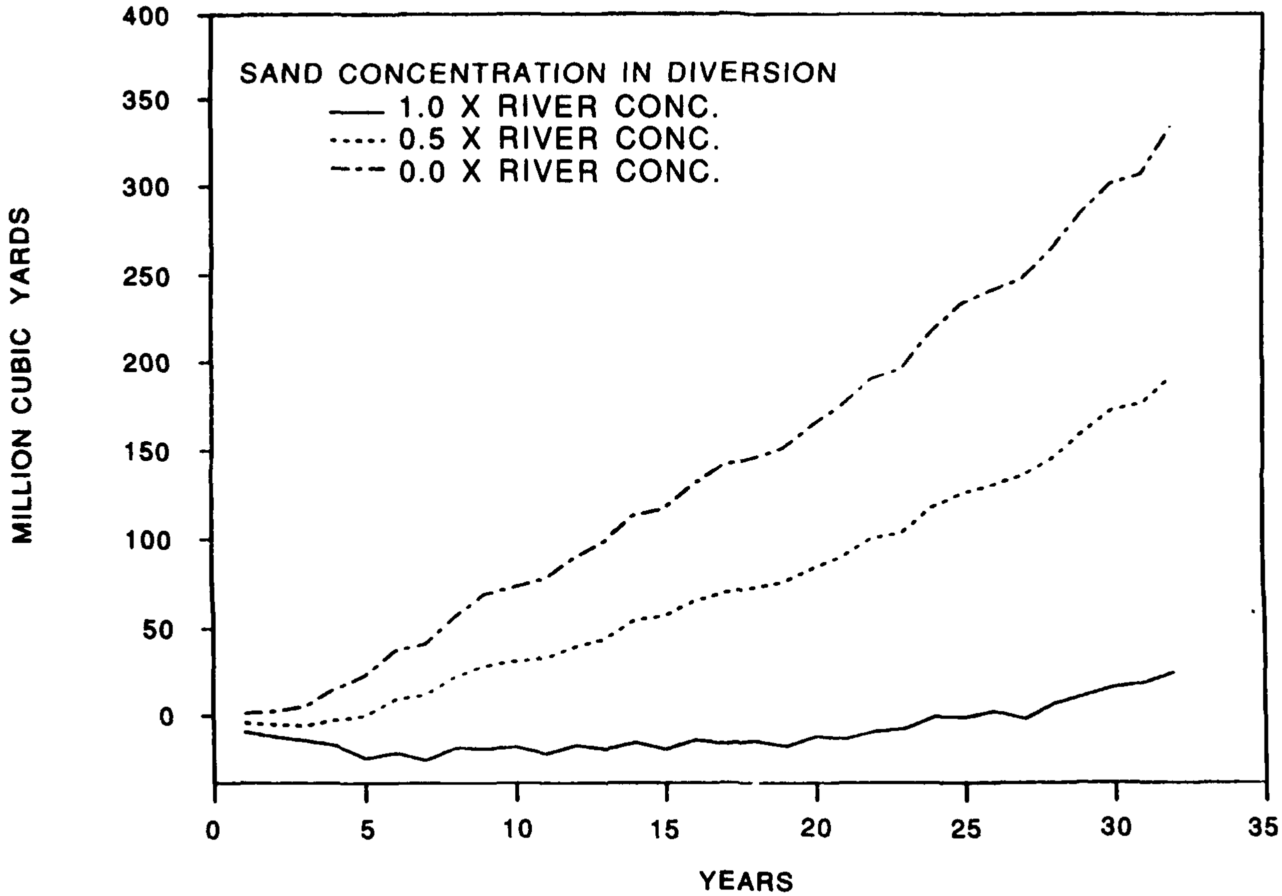


EFFECT OF 70% DIVERSION AT MILE 81
AND CLOSED PASSES ON DREDGING
IN EAST JETTY TO RESERVE

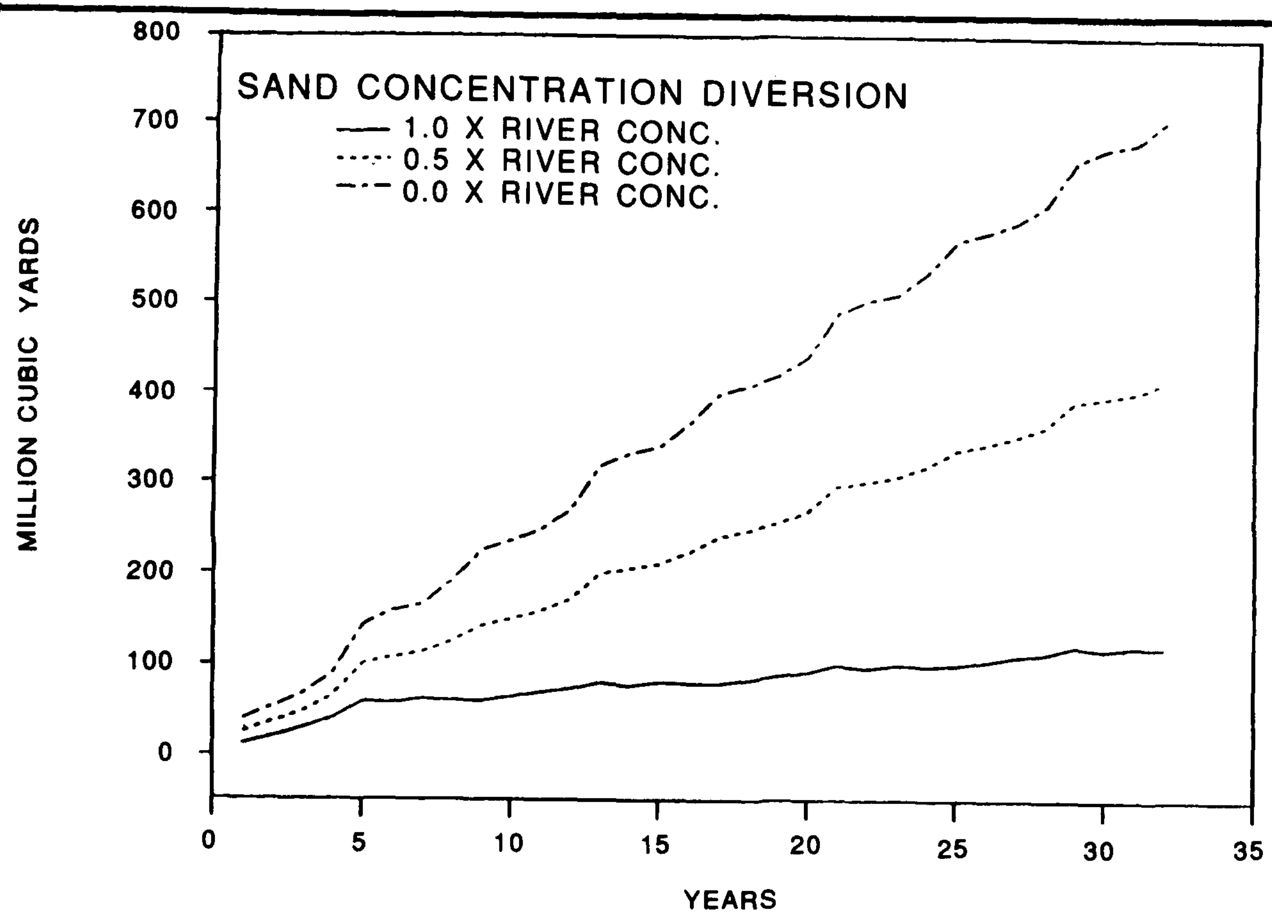


EFFECT OF 70% DIVERSION AT MILE 59
AND CLOSED PASSES ON DREDGING
IN EAST JETTY TO RESERVE

PLATE 24

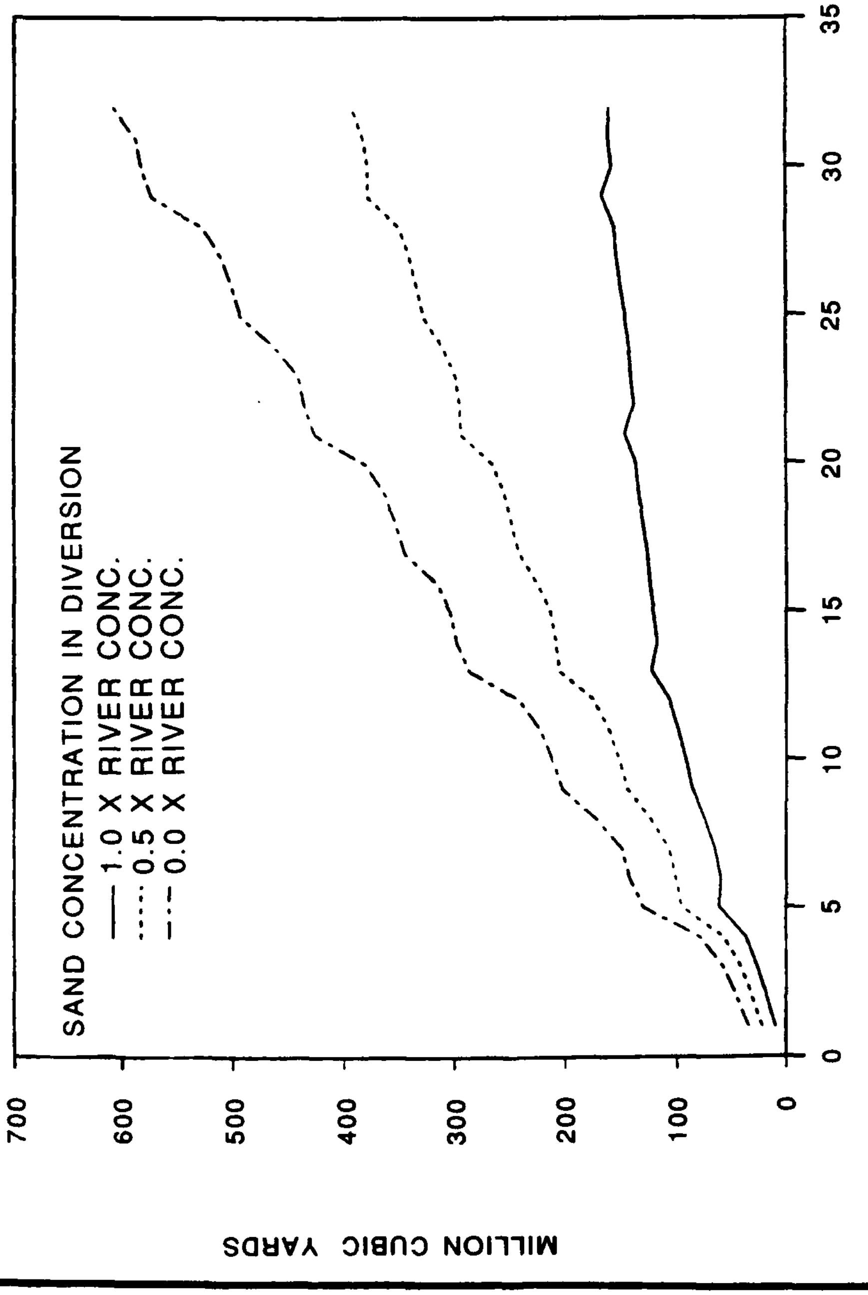


EFFECT OF 70% DIVERSION AT MILE 44
AND CLOSED PASSES ON DREDGING
IN EAST JETTY TO RESERVE

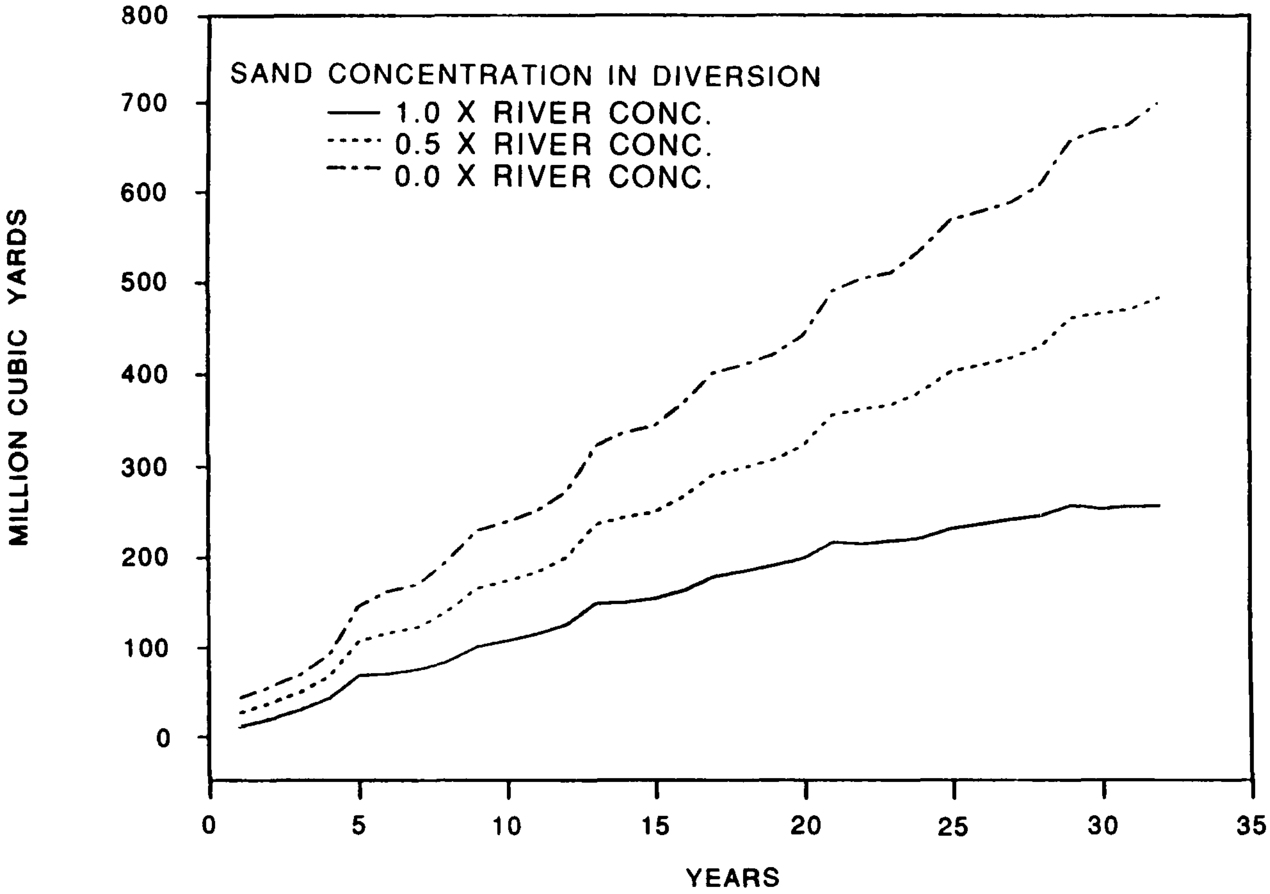


EFFECT 70% DIVERSION AT MILE 81
ON DREDGING AND DEPOSITION
IN EAST JETTY TO RESERVE

PLATE 26

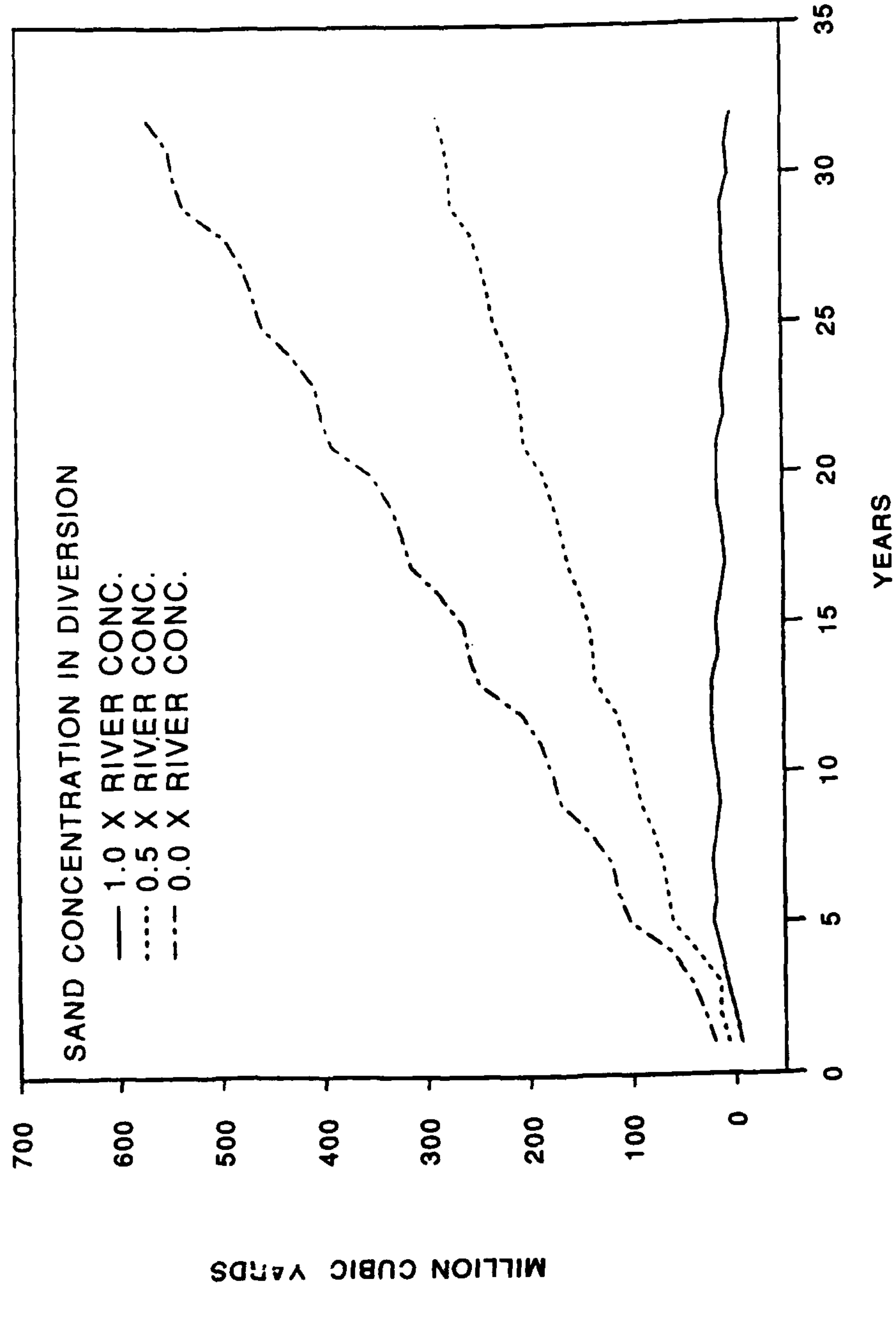


EFFECT OF 70% DIVERSION AT MILE 59 ON DREDGING AND DEPOSITION IN EAST JETTY TO RESERVE

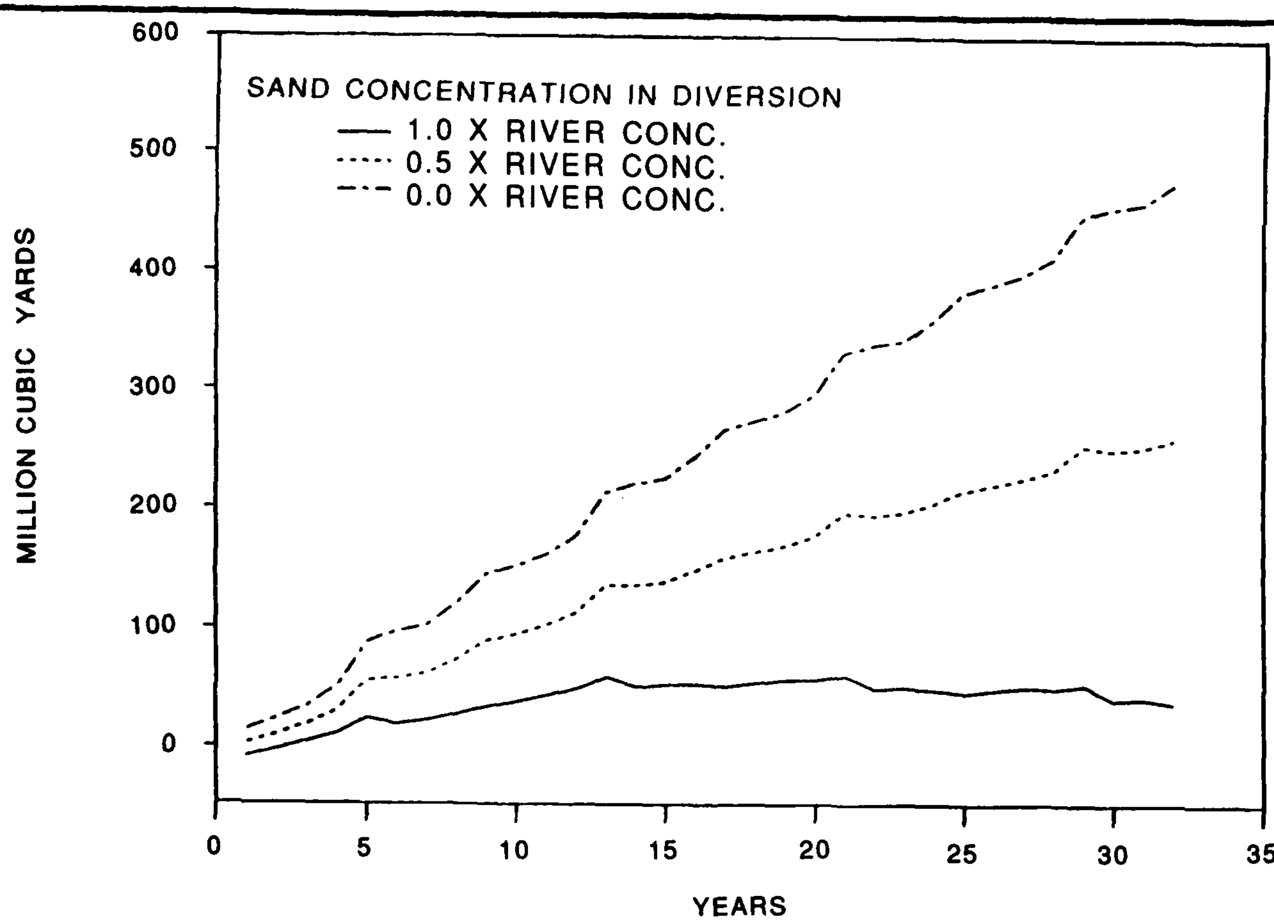


EFFECT OF 70% DIVERSION AT MILE 44 ON DREDGING AND DEPOSITION IN EAST JETTY TO RESERVE

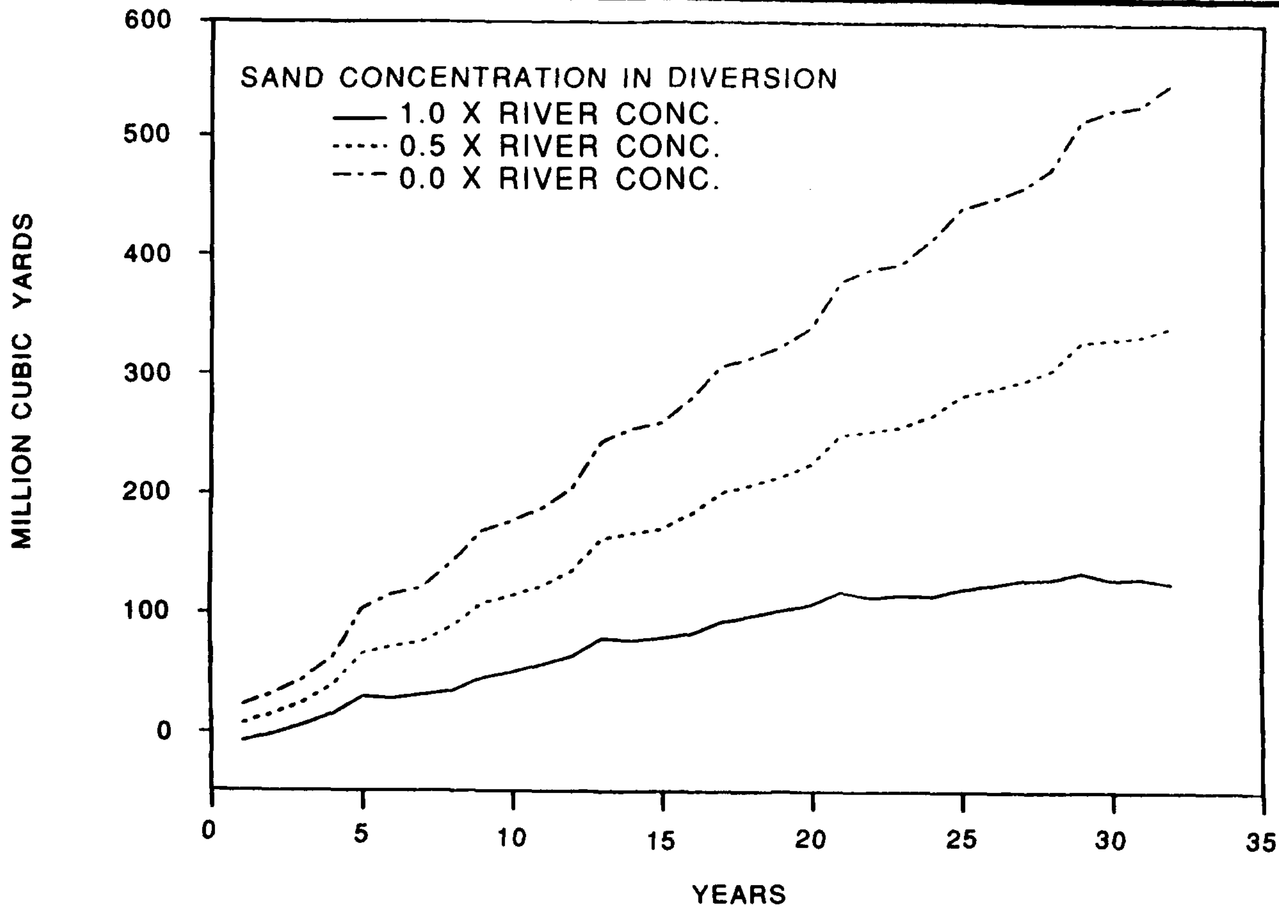
PLATE 28



EFFECT OF 70% DIVERSION AT MILE 81
AND CLOSED PASSES ON DREDGING AND
DEPOSITION IN EAST JETTY TO RESERVE



EFFECT OF 70% DIVERSION AT MILE 59 AND CLOSED PASSES ON DREDGING AND DEPOSITION IN EAST JETTY TO RESERVE



EFFECT OF 70% DIVERSION AT MILE 44 AND CLOSED PASSES ON DREDGING AND DEPOSITION IN EAST JETTY TO RESERVE

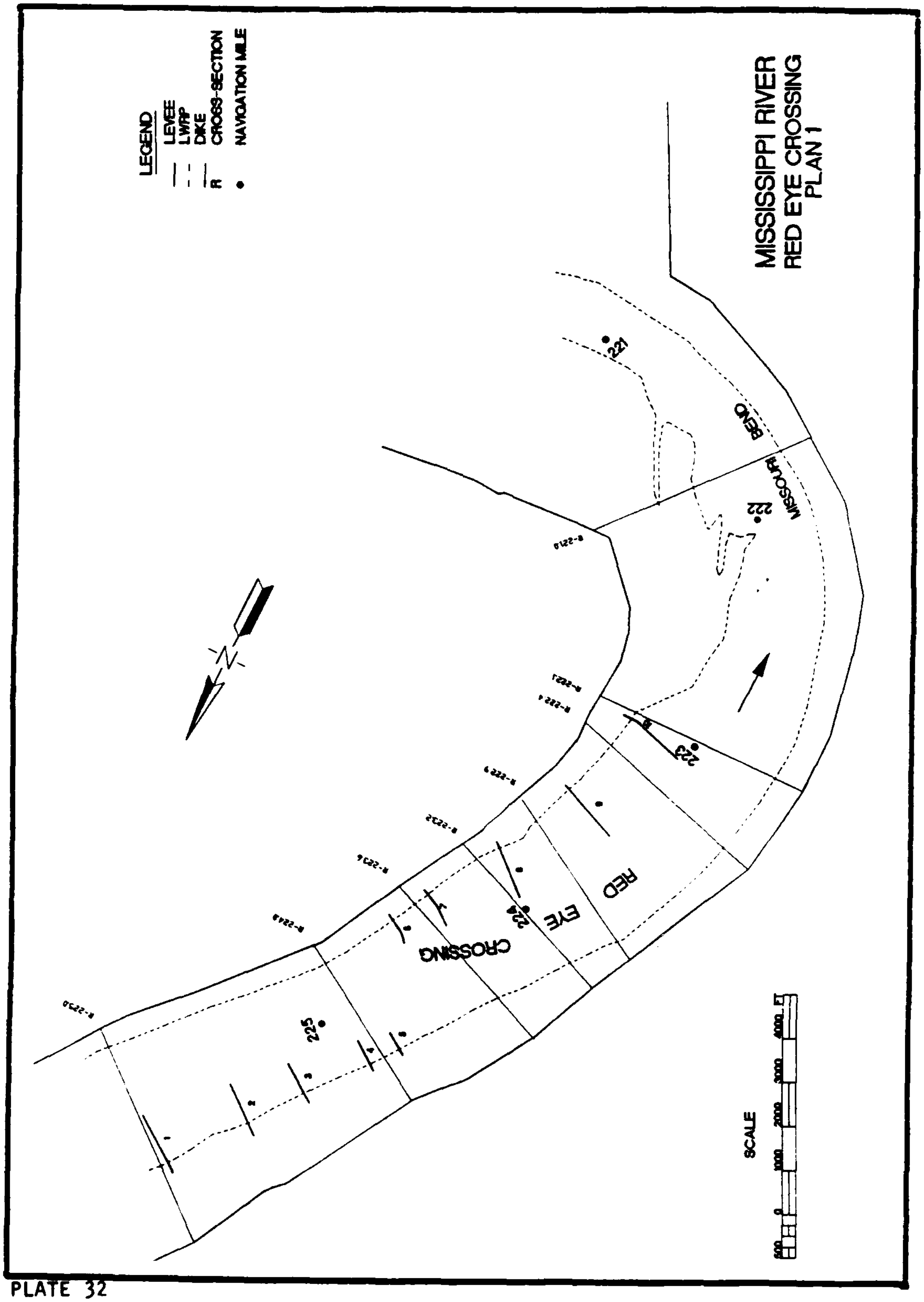

LEGEND
LEVEE
LWRP
DIKE
R CROSS-SECTION
NAVIGATION MILE
MISSISSIPPI RIVER
RED EYE CROSSING
PLAN 1
221
BEND
MISSOURI
222
223
RED
EYE
CROSSING
224
225
SCALE
0
1000
2000
3000
4000 FT

PLATE 32

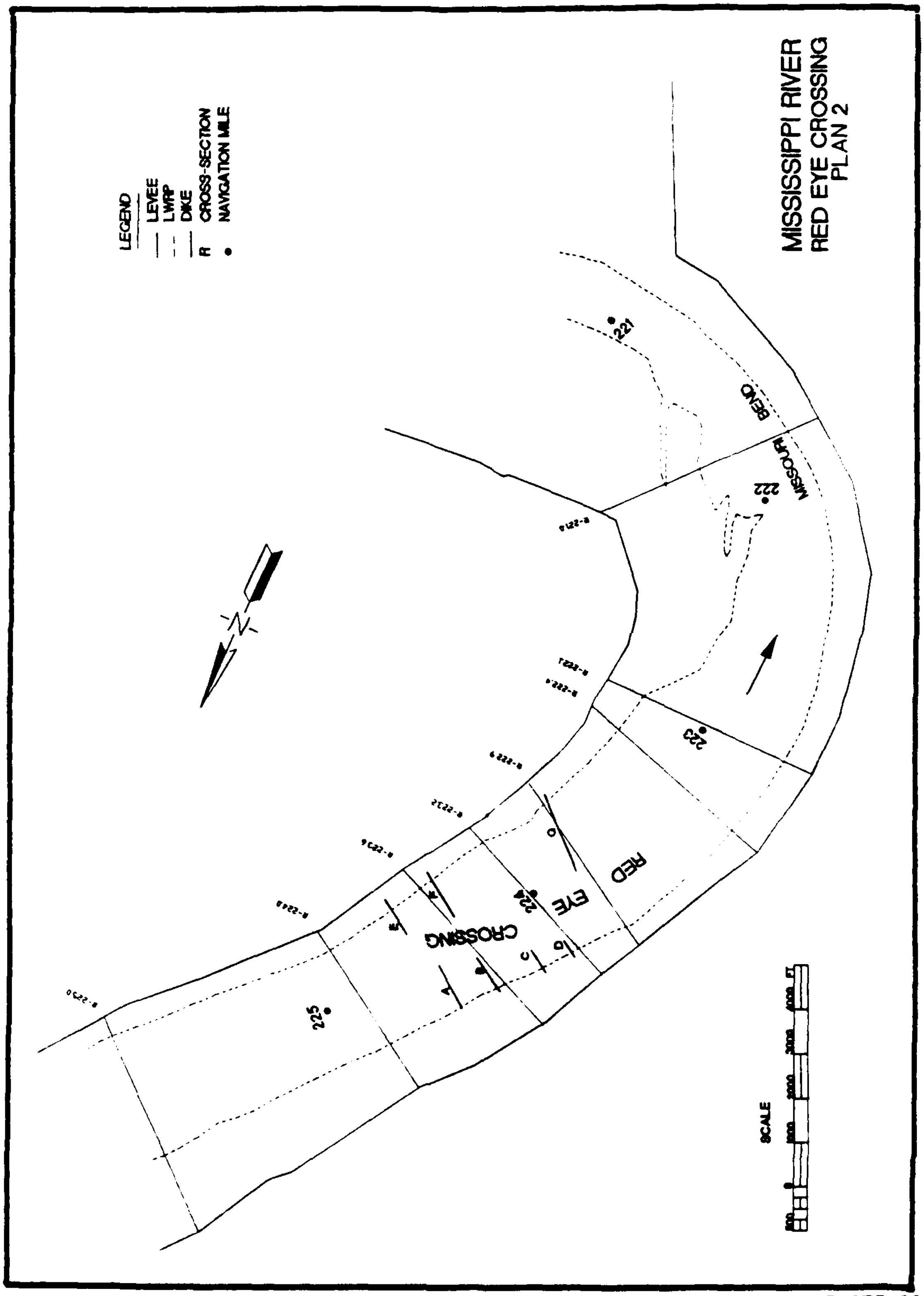

MISSISSIPPI RIVER
RED EYE CROSSING
PLAN 2
LEGEND
LEVEE
LWRP
DIKE
R CROSS-SECTION
NAVIGATION MILE
MISSOURI BEND
RED EYE CROSSING
221
222
223
224
225
SCALE

PLATE 33

APPENDIX A: DESCRIPTION OF TABS-1 GENERALIZED NUMERICAL MODEL

1. The computer program TABS-1 calculates water-surface profiles and changes in the streambed profile. Water velocity, water depth, energy slope, sediment load, gradation of the sediment load, and gradation of the bed surface are also computed. Water-surface profile and sediment movement calculations are fully coupled using an explicit computation scheme. First, the conservation of energy equation is solved to determine the water-surface profile and pertinent hydraulic parameters (velocity, depth, width, and slope) at each cross section along the study reach:

$$\frac{\partial H}{\partial X} + \frac{\partial \left(\alpha \frac{V^2}{2g} \right)}{\partial X} = S \tag{A1}$$

where

H = water-surface elevation

X = direction of flow

α = coefficient for the horizontal distribution of velocity

V = average flow velocity

g = acceleration due to gravity

S = slope of energy line

In addition, the continuity of sediment material is expressed by

$$\frac{\partial G}{\partial X} + B \cdot \frac{\partial y_s}{\partial t} = q_s \tag{A2}$$

where

G = rate of sediment movement, ft^3/day

X = distance in direction of flow, ft

B = width of movable bed, ft

y_s = change in bed surface elevation, ft

t = time, days

q_s = lateral inflow of sediment, ft^3/ft/day

The third equation relates the rate of sediment movement, G, to hydraulic parameters as follows:

$$G = f(V, y, B, S, T, d_{eff}, d_{si}, P_i) \tag{A3}$$

where

y = effective depth of flow

T = water temperature

d_{eff} = effective grain size of sediment in size class i

d_{si} = geometric mean of class interval

P_i = percentage of i^{th} size class in the bed

2. The numerical technique used to solve Equation A1 is commonly called the Standard Step Method. Equation A2 has both time and space domains. An explicit form of a six-point finite difference scheme is utilized. Several equations of the form of Equation A3 are available. These transport capacity equations are empirical and G is determined analytically.

3. Equation A2 is the only explicit equation, but it controls the entire analysis by imposing stability constraints. Several different computation schemes were tested, and the six-point scheme proved the most stable. No stability criteria have been developed for this scheme. The rule of thumb is to observe the amount of bed change during a single computation interval and reduce the computation time until that bed change is tolerable.

4. Oscillation in the bed elevation is a key factor in selecting a suitable computation interval. The computation time interval must be made short enough to eliminate oscillation. On the other hand, computer time increases as the computation interval decreases. The proper value to use is determined by successive approximations, running test cases, and observing the amount of bed change.

5. Several supporting equations are required in transforming the field data for the computer analysis. The Manning equation is used to evaluate friction loss. Average geometric properties are combined, using an average end area approach, into an average conveyance for the reach. Manning's roughness coefficients are entered for the channel and both overbanks and may be changed with distance along the channel, discharge, or stage. Contraction and

expansion losses are calculated as "other" losses by multiplying a coefficient times the change in velocity head. All geometric properties are calculated from cross-section coordinates.

6. Only subcritical flow may be analyzed in the computer program; however, zones of critical or supercritical flow may occur within the study reach. The program treats supercritical zones as "critical" for determination of water-surface elevation, but calculates hydraulic parameters for sediment transport based on normal depth. Critical depth in a section with both channel and overbank is defined as the minimum specific energy for that section assuming a level water surface. Starting water-surface elevations can be input as a rating curve with stage and discharge, or stage can be set for each specific time interval. Steady-state conditions are assumed for each time interval, although the discharge may be changed to account for tributary inflow. A hydrograph is simulated by creating a histograph of steady-state discharges, using small time intervals when discharge variations are great and longer time intervals when changes in water and sediment discharges are small.

7. In some cases the temperature of water can be an important parameter in sediment transport and, consequently, may be prescribed with each water discharge in the hydrograph. Flexibility of input permits a value to be entered as needed to change from a previous entry.

8. Geometry is input into the numerical model as a series of cross sections similar to the widely used HEC-2 backwater program (US Army Engineer Hydrologic Engineering Center 1990). A portion of the cross section is designated as movable and a dredging template may also be specified. Spacing of cross sections is somewhat more critical for TABS-1 than it is for HEC-2 because of numerical stability problems. Long reach lengths are desirable because reach length and computation interval are related. Very short time intervals may be required if excessive bed changes occur within a specific reach. No special provisions are available to calculate head losses at bridges. The contracted opening may be modeled such that scour and deposition are simulated during the passing of a flood event, but calculated results must be interpreted with the aid of a great deal of engineering judgment and sensitivity analysis.

9. Four different sediment properties are required: (a) the total concentration of suspended and bed loads, (b) grain-size distribution for the total concentration, (c) grain-size distribution for sediment in the

streambed, and (d) unit weight of deposits. A wide range of sediment material may be accommodated in the transport calculations (0.004 mm to 64 mm). The maximum size of material may exceed 64 mm so far as armoring is concerned. However, the basic assumption that the supply of large material just equals the amount transported away during each flood permits the elimination of the sizes from the transport function.

10. The usefulness of a calculation technique depends a great deal upon the coefficients which must be supplied. As in HEC-2, Manning's n values, contraction coefficients, and expansion coefficients must be provided to accomplish the water-surface profile calculations. Several other coefficients are required for sediment calculations as follows:

- a. The specific gravity and shape of sediment particles must be specified.
- b. The bed shear stress at which silt or clay particles begin to move and deposit are required coefficients.
- c. The unit weight of silt, clay, and sand deposits is somewhat like a coefficient because of the difficulty in measuring. Also, the density changes with time.

11. All of the sediment-related coefficients have default values because sediment data seem to be much more scarce than hydraulic data. There are fewer sources for generalized coefficients. All of the default values should be replaced by field data where possible, and the input data are structured for such a process.

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