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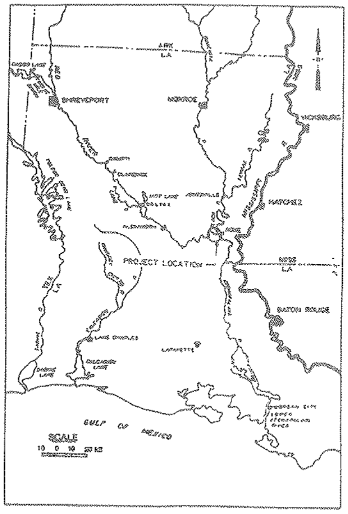

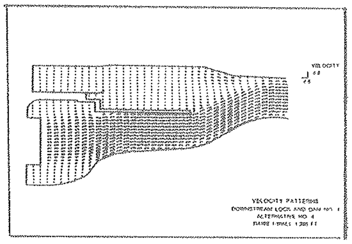

HYDRAULICS LABORATORY

TECHNICAL REPORT HL-88-15

RED RIVER WATERWAY SEDIMENTATION STUDY DOWNSTREAM FROM LOCK AND DAM NO. 1

Numerical Model Investigation

by

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Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

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TABS-2 computer program

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

The effect of recently constructed and proposed channel improvements on sedimentation in the Red River downstream from Lock and Dam No. 1 were investigated. A one-dimensional numerical model (HEC-6) was used to evaluate the effect of contraction works on dredging requirements in the navigation channel. A two-dimensional numerical model (TABS-2) was used to evaluate proposals to reduce deposition in the downstream lock approach channel at Lock and Dam No. 1. Recommendations were made to reduce sediment problems in the study reach.

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PREFACE

The numerical model investigation of the Red River downstream from Lock and Dam No. 1, reported herein, was conducted at the US Army Engineer Waterways Experiment Station (WES) at the request of the US Army Engineer District, Vicksburg (LMK).

The investigation was conducted during the period October 1984 to August 1985 by personnel of the Hydraulics Laboratory at WES under the direction of Mr. F. A. Herrmann, Jr., Chief of the Hydraulics Laboratory, and Mr. M. B. Boyd, Chief of the Hydraulic Analysis Division (HAD). Mr. W. A. Thomas of the Math Modeling Group (MMG), HAD, provided general guidance and review. The Project Engineer and author of this report was Mr. R. R. Copeland, MMG. Mr. Thomas assisted in the report preparation. This report was edited by Mrs. Marsha Gay, Information Technology Laboratory.

During the course of this study, close working contact was maintained with Messrs. Nolan Raphelt and Charles Little of the Engineering Division, LMK, who were conducting a numerical model investigation upstream from Lock and Dam No. 1, to ensure consistency in assumptions and modeling technique for the two studies. Mr. Raphelt also served as the coordinating engineer for LMK, providing required data and technical assistance. During this investigation, many representatives from both engineering staffs attended several meetings at WES and LMK to discuss progress of this investigation and others related to the Red River Waterway.

COL Dwayne G. Lee, CE, is the Commander and Director of WES. Dr. Robert W. Whalin is the Technical Director.

CONTENTS

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
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
pounds (force)-second per square foot	47.88026	pascals-second
tons (2,000 pounds, mass)	907.1847	kilograms

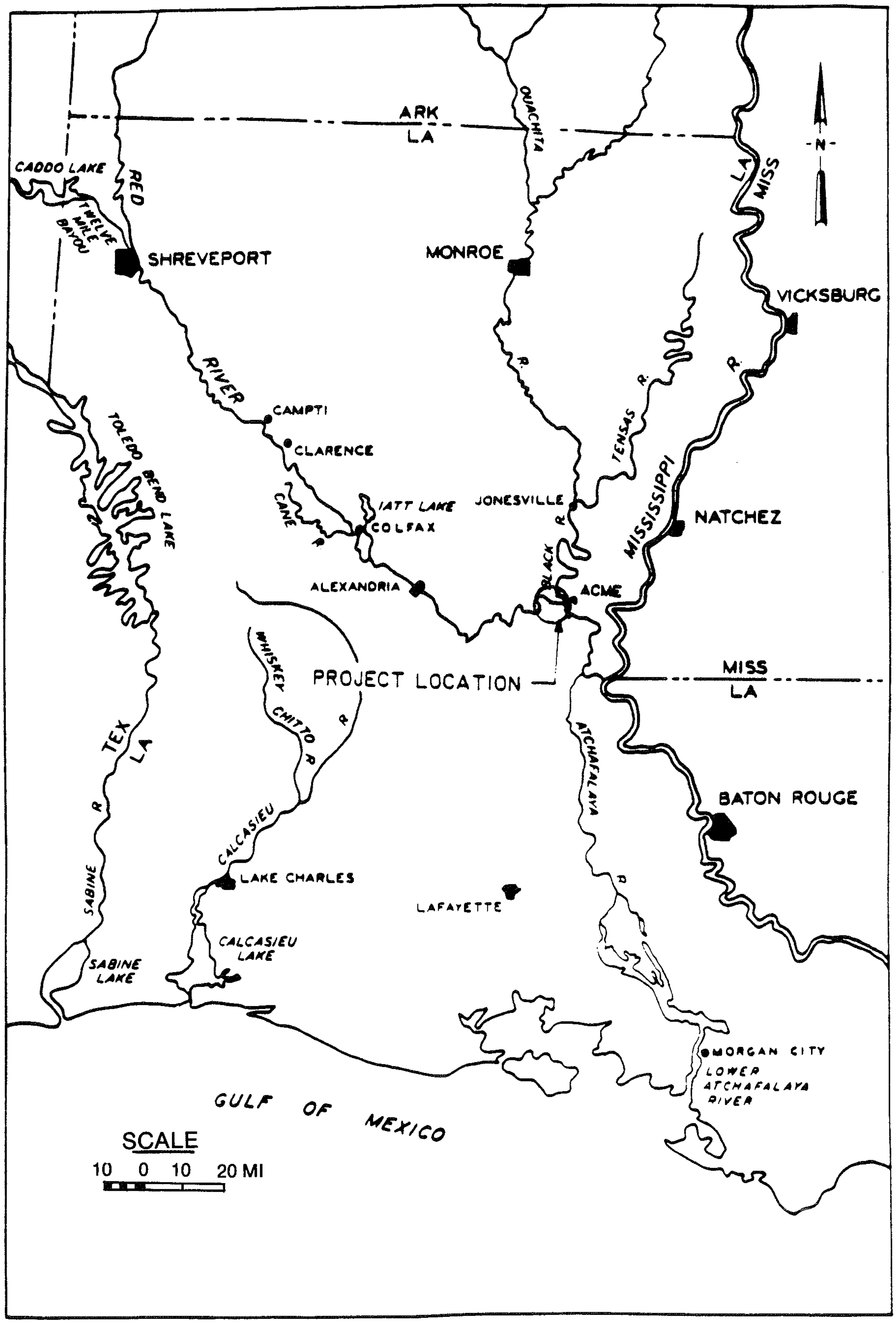

Figure 1. Location map

RED RIVER WATERWAY SEDIMENTATION STUDY DOWNSTREAM FROM LOCK AND DAM NO. 1

Numerical Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Red River Waterways Project will provide a navigation route from the Mississippi River at its junction with Old River to Shreveport, Louisiana, via the Old and Red rivers. The project will provide a channel 236 miles* long, 9 ft deep, and 200 ft wide, and will include a system of 5 locks and dams to control water levels. The existing river will be realigned as necessary to develop an efficient channel, and bank stabilization and training works will be constructed to hold the newly developed channel in position.

2. Lock and Dam No. 1, the downstream navigation structure on the Red River, is located in a river cutoff approximately 43.7 miles above the confluence with the Mississippi River (Figure 1). It consists of a single lock on the left descending side of the cutoff and a 640-ft-long dam with eleven 50-ft-wide gates (Figure 2). The channel downstream from the lock and dam has a design invert elevation of 0.0.** The invert elevation in the downstream lock approach channel is -7.0 dropping to -11.0 just upstream from the lock miter gate, which has a sill elevation of -9.0. The floor elevation in the 84- by 785-ft lock chamber is -11.0. A 1,300-ft-long dike and I-wall with a crest elevation of 38.0 separate the downstream lock approach channel from the spillway exit channel. This I-wall was designed to overtop during high flows. A 700-ft-long floating guide wall runs parallel to the I-wall. The dam is designed to maintain a normal pool elevation of 40 and to pass the project flood of 255,000 cfs, which is the 100-year frequency flood.

3. Downstream from Lock and Dam No. 1 the Red River traverses the Mississippi River floodplain. The river is characterized by large

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* A table of factors for converting non-SI units of measurement to SI (metric) units of measurement is presented on page 3.

** All elevations (el) and stages cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

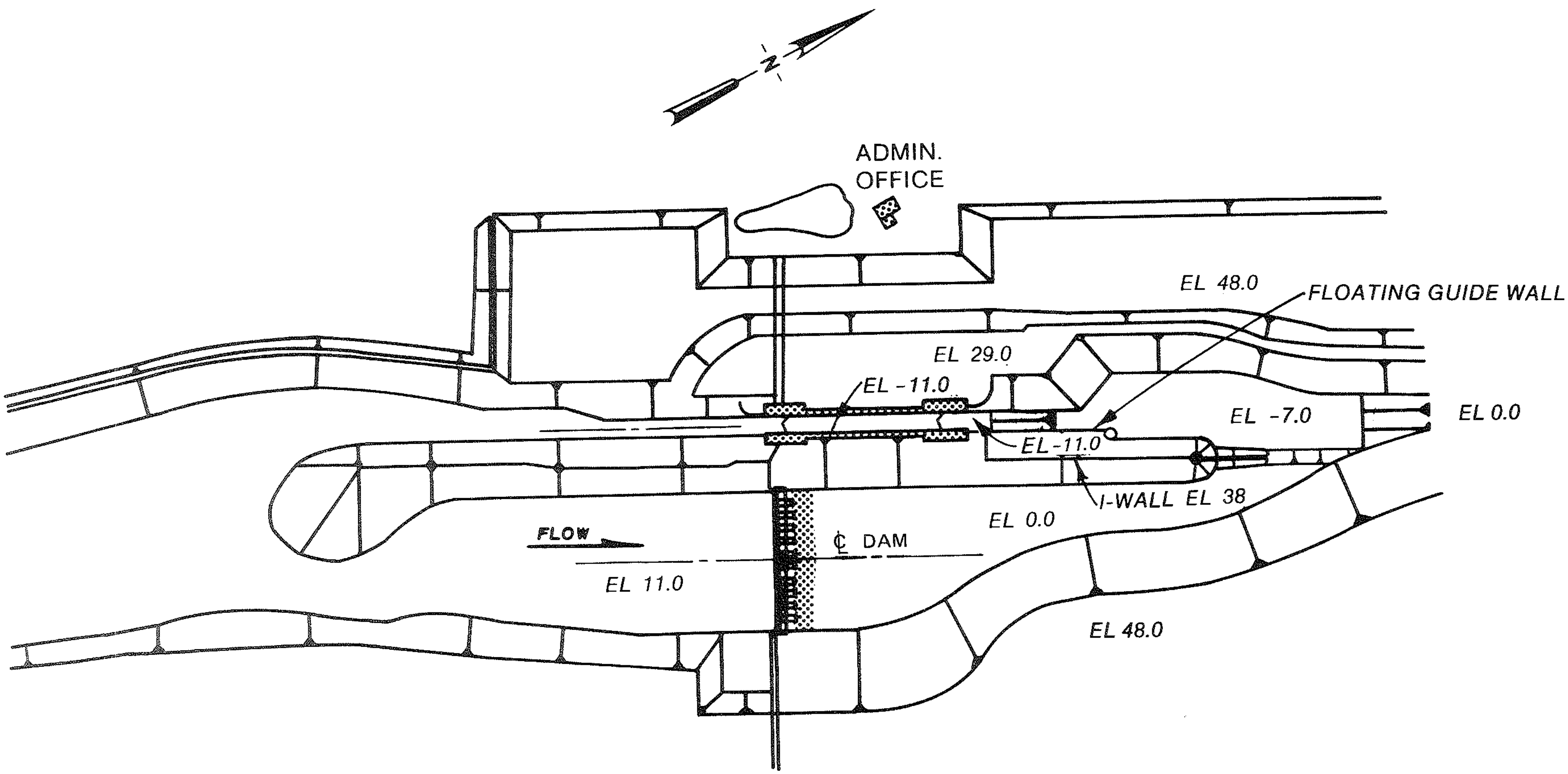


Figure 2. Red River Waterway Lock and Dam No. 1, as constructed

fluctuations in stage, shifting bed and banks, and unpredictable shoaling. The water stage in this reach is dependent not only on flow magnitude in the Red River but also on the backwater effects of flows from the Mississippi River through the Old River Diversion Channel.

Purpose

4. The purpose of this study was to evaluate the effects of channel improvements on sedimentation between Lock and Dam No. 1 and the confluence with the Black River. The effect of contracting the channel to increase sediment transport and reduce dredging requirements was studied. Design modifications to the downstream lock approach channel were studied and their effect on sediment disposition evaluated.

Scope

5. Two numerical model studies were conducted. A one-dimensional (1-D) sediment transport model, HEC-6, was used to calculate deposition, scour, and dredging quantities for various trace widths. A trace width is a designated river width that is assumed to convey all the flow. When training dikes are present, trace width is taken as the distance between the outer ends of the dikes on opposite banklines. Trace widths of 200, 300, 400, and 500 ft were tested with a 7-year hydrograph. The model calculated dredging requirements necessary to maintain a 200-ft-wide navigation channel with a 9-ft draft. The model also calculated average velocities in the contracted channel. A two-dimensional (2-D) numerical model (TABS-2) was used to evaluate possible project modifications to reduce deposition in the downstream lock approach channel. This model extended downstream from Lock and Dam No. 1 for about 15 miles. The model simulated the as-built spillway exit and lock approach channels and the I-wall between them. The effect of raising various lengths of the I-wall above the water-surface elevation was tested.

PART II: 1-D MODEL

Description

6. The NETWORK version of the HEC-6 computer program was used to develop the 1-D numerical model. The HEC-6 program (US Army Hydrologic Engineering Center 1977) produces a 1-D 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, their effect on the sediment transport capacity at cross sections, and the resulting degradation or aggradation which is assumed to occur uniformly across the entire channel cross section. The NETWORK version incorporates several modifications and expanded capabilities that have been developed at the US Army Engineer Waterways Experiment Station. A new dredging routine was added to the program to accommodate the dredging specifications for this study.

Channel Geometry

7. Cross sections for the 1-D numerical model were taken from the 1967-1968 hydrographic survey of the Red River. The primary area of interest in this study extended from Lock and Dam No. 1 (river mile 46) to the confluence of the Red and the Black rivers (river mile 34). In this reach, cross sections were located at approximately one-half-mile intervals. The model was extended to Shreveport (river mile 277) to account for possible channel storage and supply downstream from the Shreveport sediment gage, and to make use of sediment measurements at Alexandria (river mile 105) to adjust the model. Between Lock and Dam No. 1 and river mile 140, cross sections were located at approximately 2-mile intervals. Upstream from river mile 140, cross-section intervals averaged 14 miles. This geometry was used in the adjustment phase of the study in which roughness coefficients and bed material gradation were determined.

8. The model geometry was revised to represent design channel conditions with Locks and Dams Nos. 1 and 2 in place. This revision amounted to eliminating cross sections and reducing reach lengths to account for proposed cutoffs and cutoffs constructed since 1968 (US Army Engineer District (USAED),

New Orleans, 1982). No changes were made in the reach downstream from Lock and Dam No. 1 because information defining the new cross-section geometry was not available, and because the proposed cutoffs were deemed relatively insignificant with respect to change in total reach length. Cross sections just upstream from the locks and dams were taken from design drawings and represent channel geometry upstream from the lock walls. This revised geometry was used to determine the aggradation and degradation potential of the river without any contraction works.

9. The effect of various trace widths downstream from Lock and Dam No. 1 on aggradation and degradation was evaluated by restricting flow and sediment movement to the specified width, ignoring dike overtopping and overbank flows. This channel configuration was simulated in the model with frictionless vertical walls. Trace widths of 200, 300, 400, and 500 ft were tested. A more detailed study would include an accurate definition of the dikes including the sloping crest elevations and the area between dikes, accounting for deposition and increase of roughness due to vegetation. It would also include overbank areas for conveyance of flood flows.

Stage and Discharge

10. The water-surface elevation at the downstream boundary is controlled by flows in the Atchafalaya River and the Old River Control Structure Outflow Channel, and is not directly a function of discharge in the Red River. Starting water-surface elevations in the numerical model were therefore determined from the stage hydrograph at Acme, Louisiana (Black River mile 0.1). In the steady-state numerical simulations, stages at Acme for a specific day were assumed to correspond to the discharge at Alexandria for the same day, ignoring possible attenuation of the hydrograph due to storage and routing in the 71 miles between the gages.

11. Minimum pool elevations of 40 and 64 ft were assigned upstream from Locks and Dams Nos. 1 and 2, respectively. When the downstream water-surface elevation exceeded the minimum pool elevation, a head loss of 1 ft was arbitrarily assigned between the upstream and downstream cross sections.

12. A 7-year hydrograph (1975-1981) was used to evaluate the base condition and various trace widths. This hydrograph represents the most recently available data but does not necessarily represent the long-term flow or stage

averages. The maximum daily discharge in the 1975-1981 hydrograph was 124,000 cfs, which is less than a 5-year frequency event. The effect of major flood events is therefore not included. Figure 3 compares the flow durations for the 1975-1981 hydrograph and the 1938-1976 period of record (adjusted to account for construction of upstream dams). Stages at Acme were significantly lower during the 1975-1981 period than during the 1938-1976 period of record (Figure 4). This difference can be accounted for by the general degradation of the Atchafalaya River, which controls water-surface elevations at Acme. Results using the 1975-1981 hydrograph should therefore be considered relative, and not taken to represent long-term average conditions. Average annual quantities could better be determined using a long-term hydrograph (50 years) that includes extreme events and hypothetical floods.

Bed Material

13. Surface bed material gradation measurements were available at Shreveport (1977-1979), Alexandria (1971-1972; 1975-1981), and above Old River Outflow Channel and river mile 13.1 (1974). The measurements indicate that the surface bed material is highly variable. Average annual data at Alexandria are shown in Table 1. These data indicate that the bed surface generally became finer from 1971 to 1977 and then coarsened through 1981. Measurements at Shreveport (Figure 5) show significant variations in the bed surface gradations, which appear to have little or no correlation to discharge.

14. The gradation of the bed material reservoir is an input requirement for the numerical model. This reservoir represents the average gradation of the bed material to the depth of reasonably expected scour. This depth was set at 10 ft upstream from Lock and Dam No. 1 and 20 ft downstream. The model calculates an active layer thickness that is dependent on the hydraulic parameters, the gradation of the previous active layer, and the gradation of the reservoir. It is the gradation of the active layer that determines sediment transport past a cross section with respect to scour or deposition. This active or surface layer gradation may be considerably different from that of the bed material reservoir. Available bed material measurements correspond to this active layer. Measurements of the bed material reservoir were not available; therefore, the gradation was used as an adjustment parameter in the numerical model study.

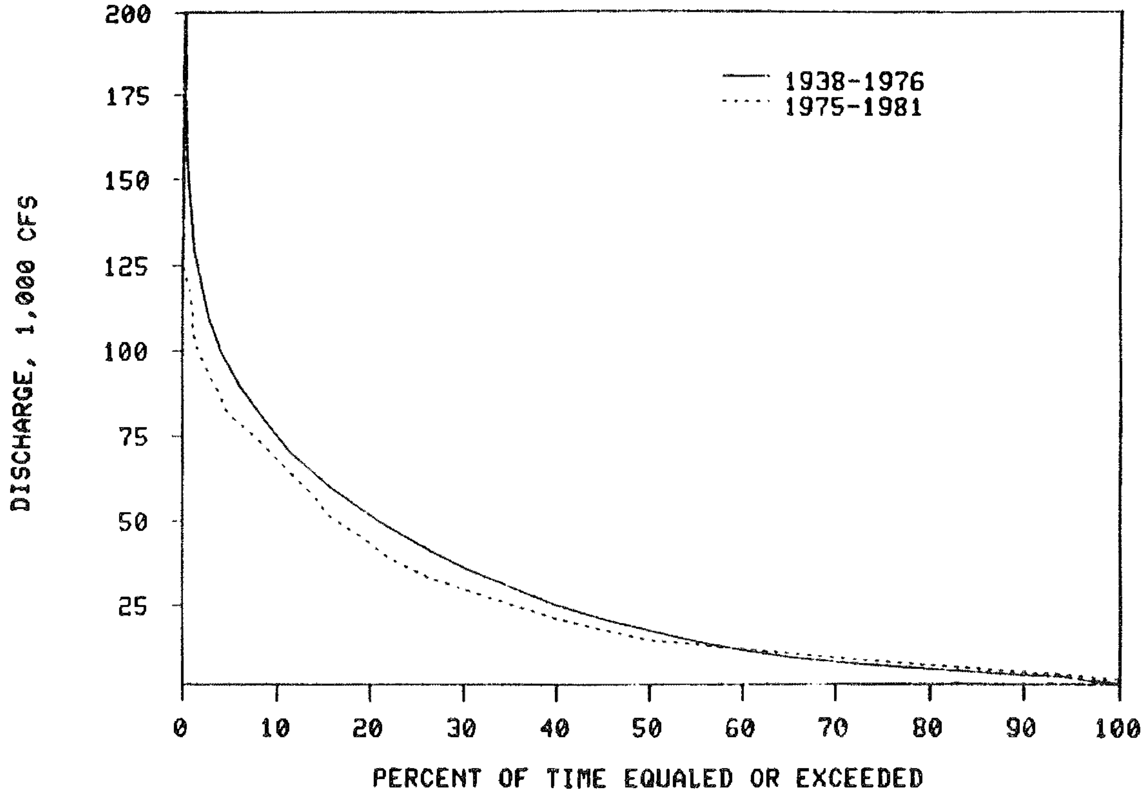


Figure 3. Flow duration curves, Alexandria

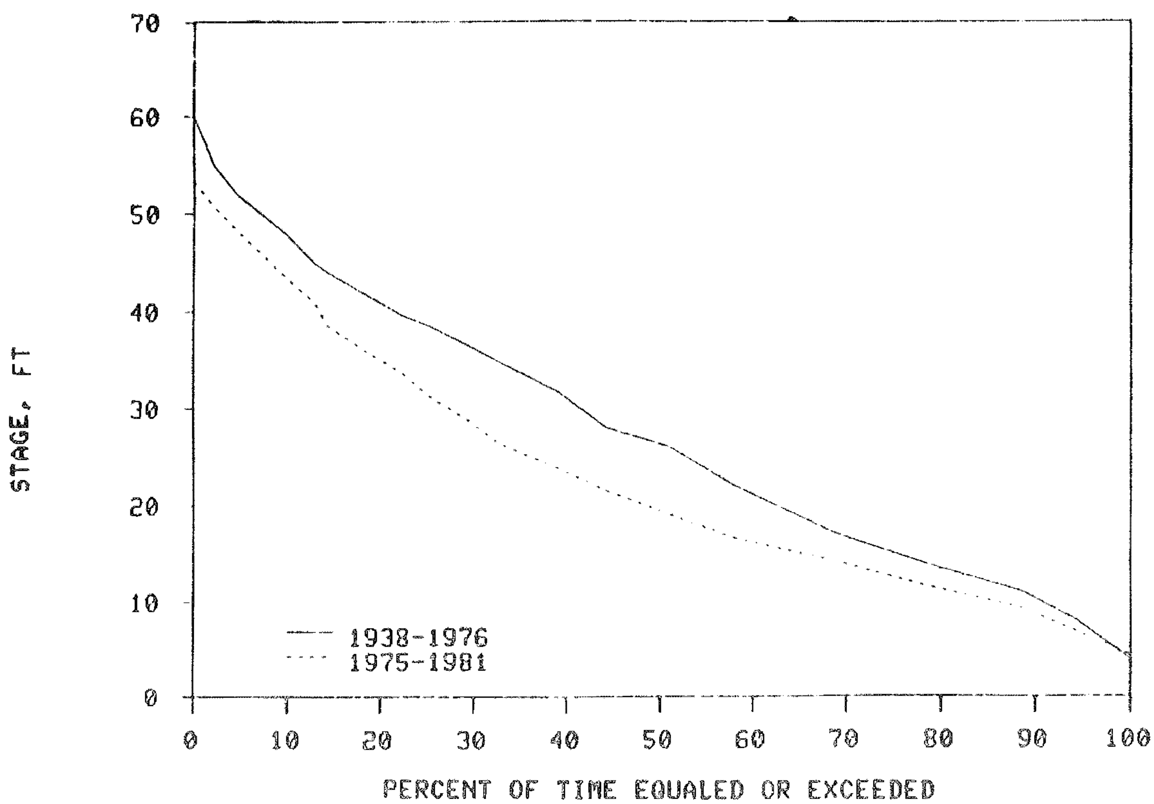
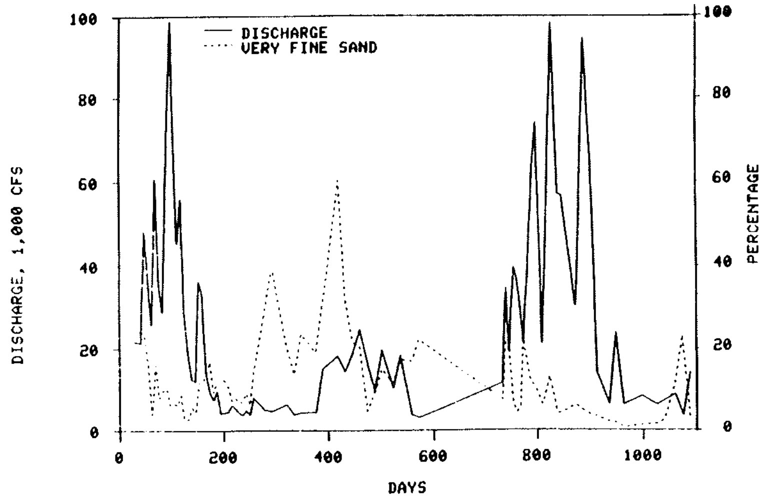
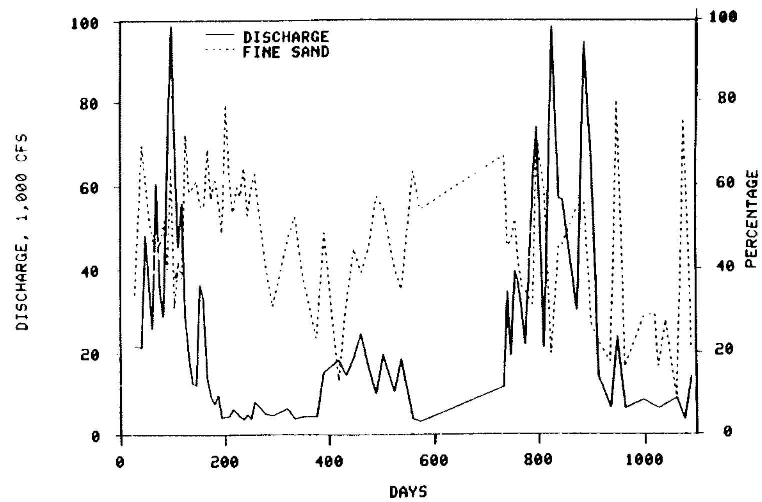


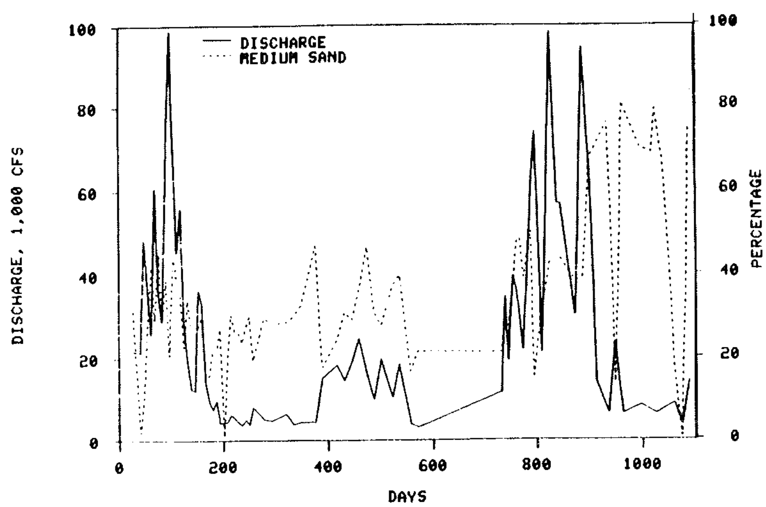
Figure 4. Stage duration curves, Black River at Acme



a. Very fine sand



b. Fine sand



c. Medium sand

Figure 5. Measured surface bed material at Shreveport, 1977-1979

Sediment Inflow

15. A summary of average annual suspended sand concentrations at Alexandria (USAED, New Orleans, 1980b) indicates that a significant decrease in sediment transport occurred on the Red River after 1975 (Figure 6). For this reason, the sediment inflow rating table used in a previous New Orleans District model, which was based on sediment measurements taken at Shreveport between 1965 and 1970 (USAED, New Orleans, 1980a), was not used in this study. The only suspended sediment measurements available at Shreveport after 1975 were in 1977 and 1978. A regression analysis of these data was used to develop a sediment inflow rating curve for very fine, fine, medium, and coarse sands. Measurements taken at discharges greater than 20,000 cfs were analyzed separately to obtain the upper portion of the rating curves. Suspended materials finer than sand were not considered in this study due to the inability of the 1-D model (which uses average values for hydraulic parameters at each cross section) to account for their deposition or scour in slack-water areas.

16. The effect of bank erosion between Shreveport and Alexandria on the sediment discharge at Alexandria was investigated. Suspended sediment measurements at Shreveport and Alexandria for 1977-1978 were compared (Figure 7). This comparison showed that there was actually a decrease in sand load at Alexandria. This decrease is attributed to hydraulic parameters. This analysis did not support the notion that the sand load at Alexandria should be increased due to contributions from bank erosion.

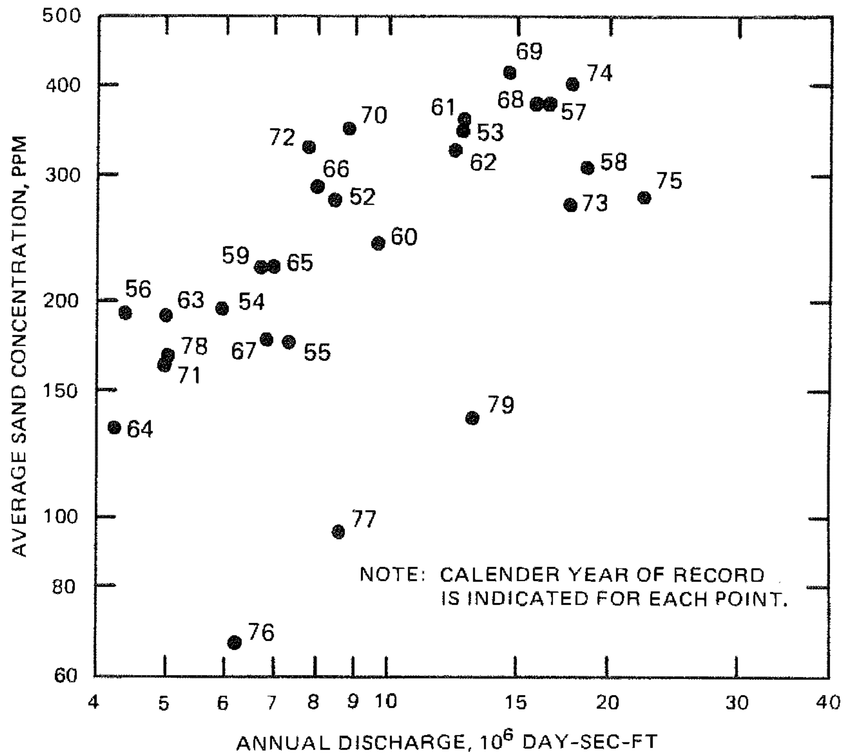


Figure 6. Average annual suspended sand concentration at Alexandria

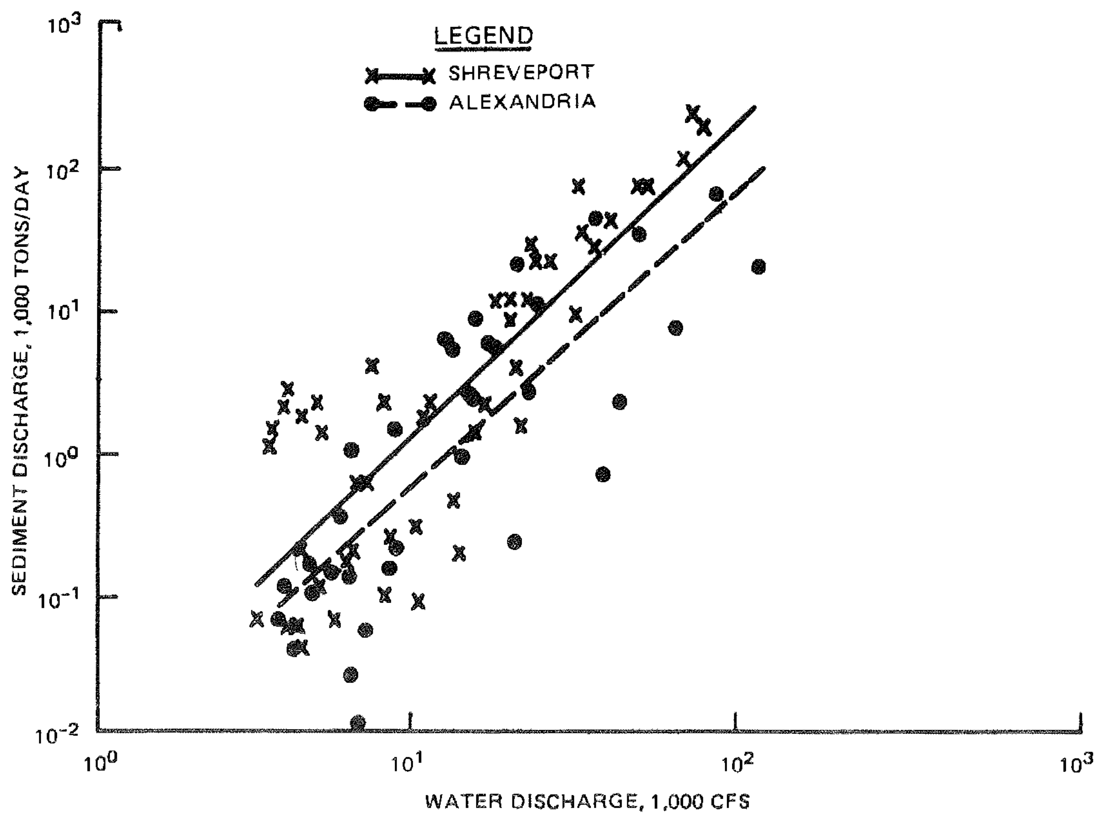


Figure 7. Suspended sand measurement at Alexandria and Shreveport, 1977–1978

PART III: 1-D MODEL ADJUSTMENT

Water-Surface Adjustment

17. Roughness coefficients were adjusted in the numerical model until calculated water-surface profiles corresponded to measured stages at five gages: Moncla (river mile 67.7), Alexandria (river mile 104.9), Boyce (river mile 125.4), Colfax (river mile 140.5), and Shreveport (river mile 277). The 3 April 1968-30 August 1969 hydrograph was used to develop a rating curve of Manning's n versus discharge. A hydrograph was used so that bed changes that occur with the rising and falling of flood stages would be considered. These changes are especially important when considering low flow discharges where the water-surface elevation is influenced by the bed elevation of river crossings. Initial bed gradations in the numerical model were taken from a previous study (USAED, New Orleans, 1980a). After a new bed material gradation was determined in this study, the water-surface profile adjustment was repeated and the final roughness coefficients determined (Table 2). Adjusted roughness coefficients upstream from river mile 182 were significantly lower than downstream values. This difference is attributed to a generally wider channel and to the increased reach lengths between cross sections in the numerical model. In this reach, the n values include a geometry adjustment factor and should not be considered transferable to more detailed studies. Calculated water-surface elevations are compared to measured stages in Figure 8.

Sediment Transport Adjustment

18. The bed material gradation in the numerical model was adjusted so that the calculated sediment transport at Alexandria corresponded to measured data. This process also served as an adjustment of the transport function, which in this model was that developed by Toffaleti. This function is deemed appropriate because Toffaleti used data at Alexandria in the derivation of his equations.

19. Initially, an iterative process was used to estimate an appropriate bed material gradation. Measured bed data at Shreveport and Alexandria were used to develop an initial bed gradation. After a 7-year hydrograph was run,

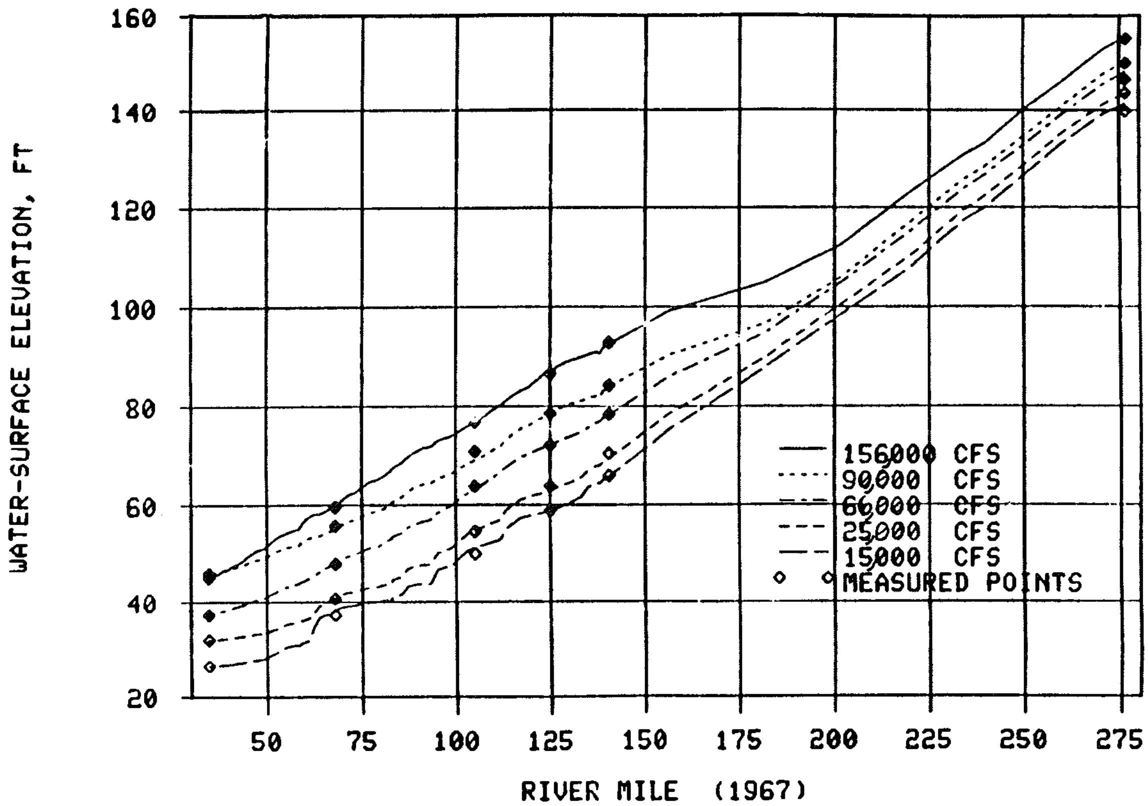


Figure 8. Calculated and measured water-surface elevations

initial and final bed gradations were compared. If the difference was greater than 5 percent, another run was made using the calculated bed gradation as initial conditions. This process was repeated until a stable gradation was determined. Using this technique produced a calculated sand transport at Alexandria that was too high. It was concluded that this process would be successful only if percentages of medium and coarse sand were essentially correct in the beginning. These size classes move too slowly to produce a significant change in the bed material reservoir in a reasonable time period.

20. A second procedure was used in which the bed material measurements taken during high flows at Shreveport and above Old River Outflow Channel were used to develop a single gradation for the entire reach (Figure 9). The Alexandria station is located in a pool with a revetted bank. Pools are characterized by highly variable lateral velocity distributions and therefore variable lateral bed material gradations. Measurements taken from the bar adjacent to the pool would be expected to be finer than measurements taken at the Shreveport and above Old River Outflow Channel gages, which are not located in pools. Data from crossings where lateral velocity distribution is

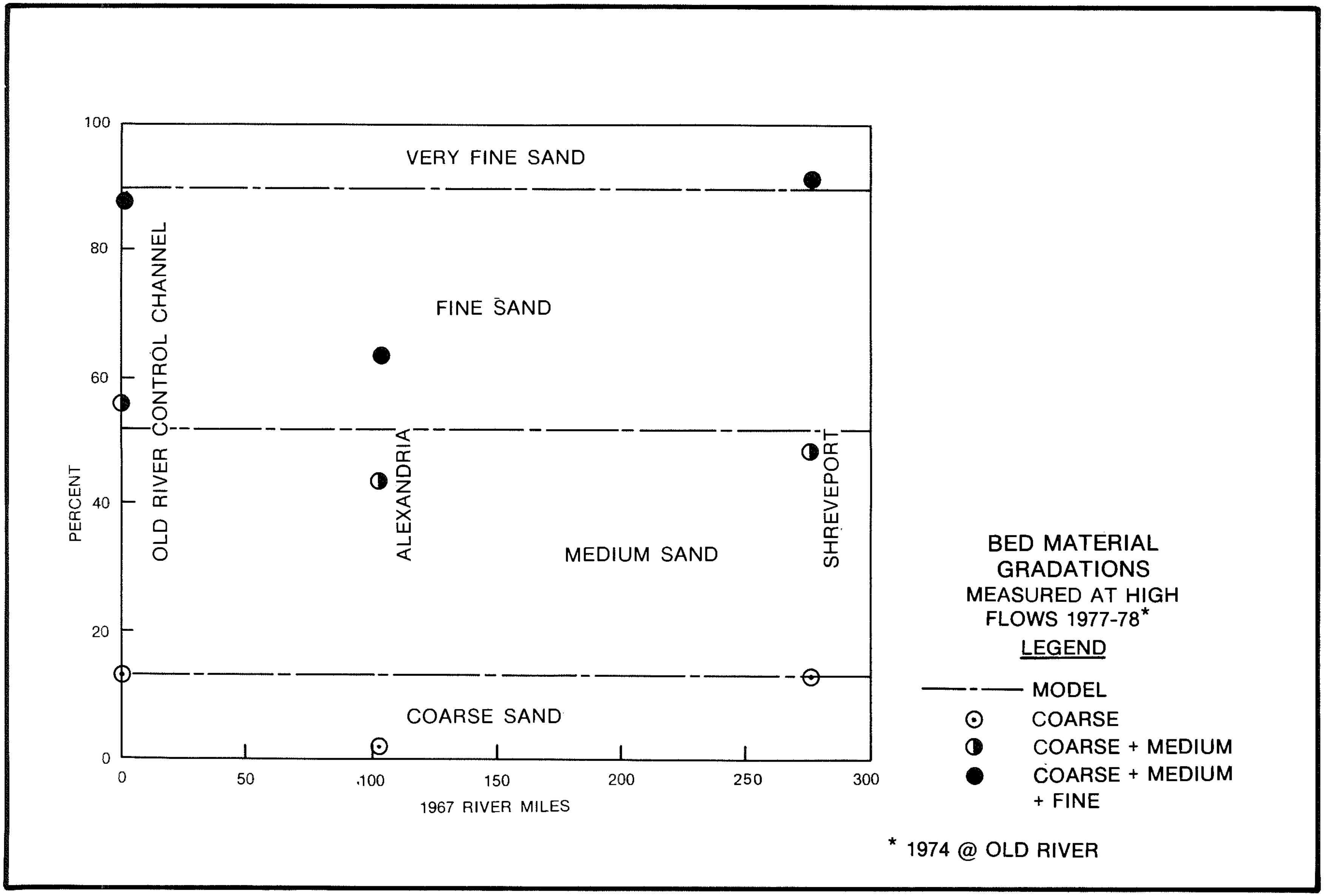
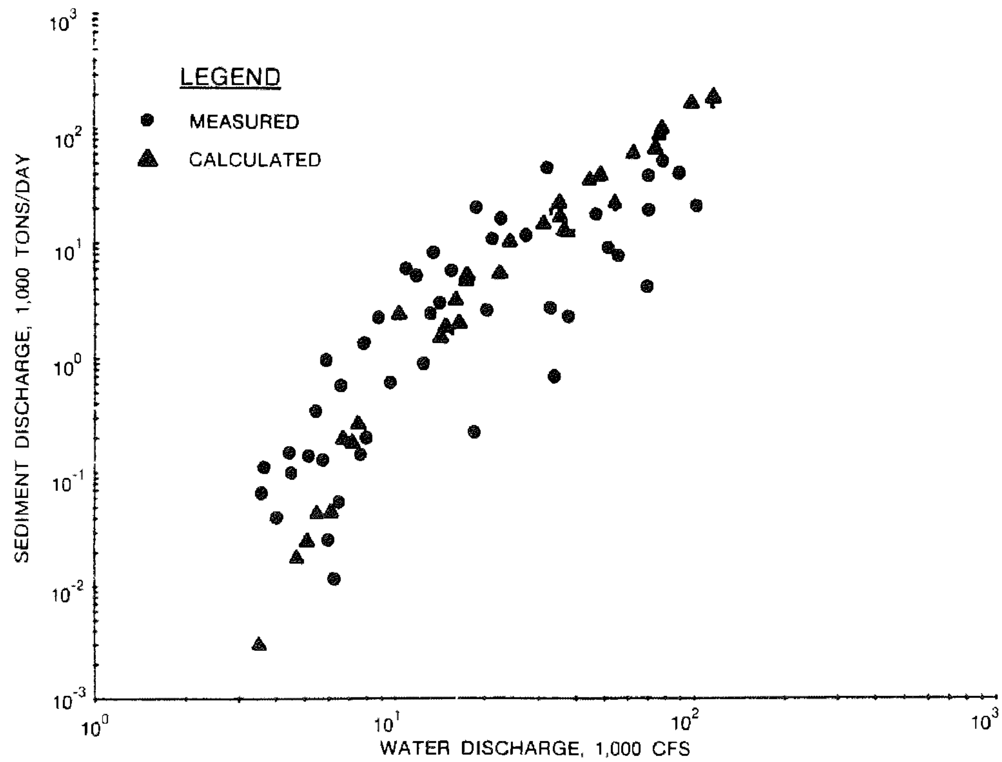
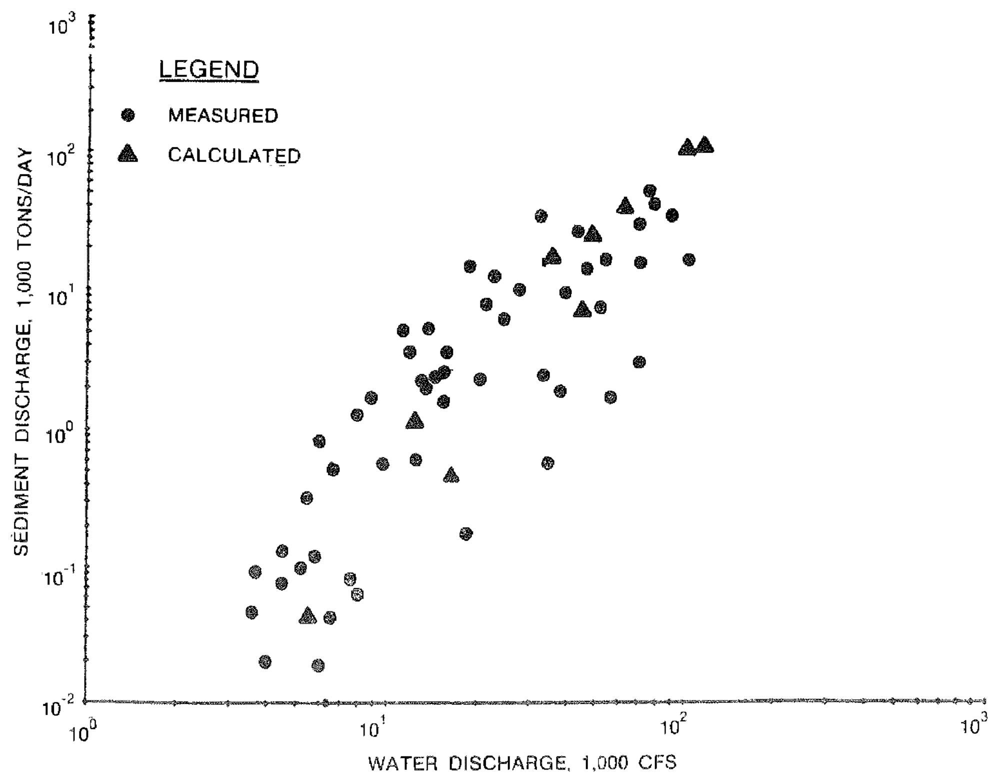


Figure 9. Bed material gradations

more constant give a better representation of bed material for purposes of the model because average velocities are used in the model to determine transport. High flow measurements are expected to represent the gradation of the bed material reservoir more accurately because more mixing occurs during highly turbulent flows. Calculated sediment transport at Alexandria, using this bed material gradation, satisfactorily reproduced measured data (Figure 10). Simulation of the sediment discharge at Alexandria and water-surface elevations constituted successful adjustment of the numerical model.

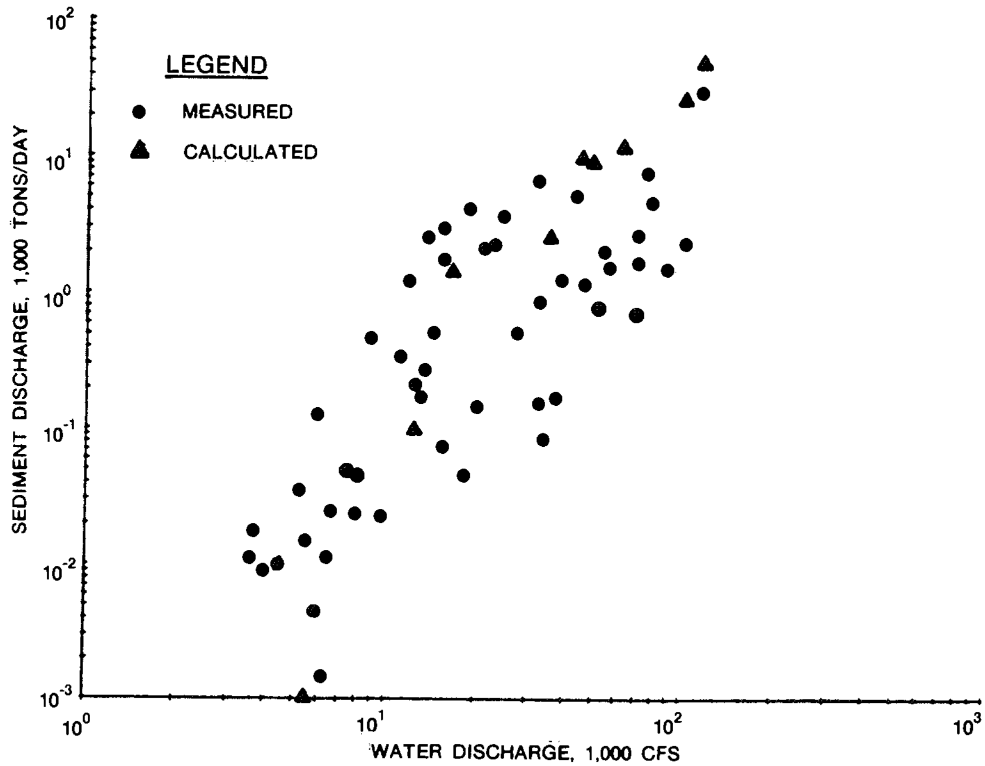


a. Sand transport at Alexandria, 1977-1979

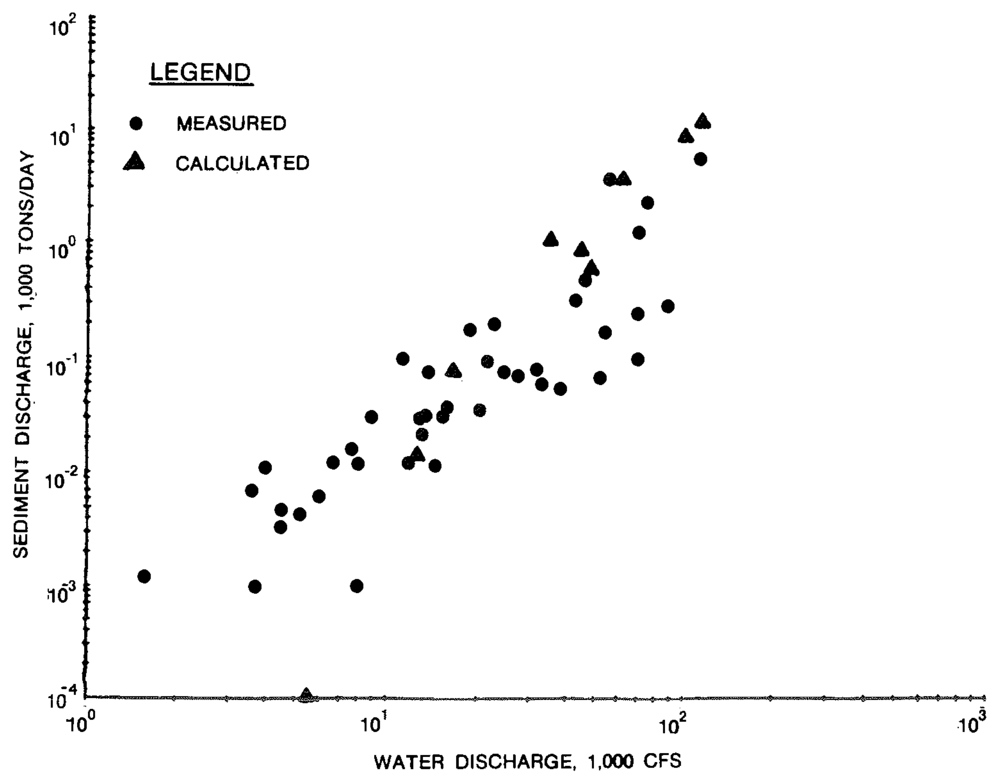


b. Very fine sand transport at Alexandria, 1977-1979

Figure 10. Measured and calculated sediment transport (Continued)



c. Fine sand transport at Alexandria, 1977-1979



d. Medium sand transport at Alexandria, 1977-1979

Figure 10. (Concluded)

PART IV: 1-D MODEL RESULTS

Base Conditions

21. Channel improvements were incorporated into the adjusted numerical model to establish a base condition for the trace width tests. The improvements included Locks and Dams Nos. 1 and 2 and existing and proposed cutoffs upstream from Lock and Dam No. 1. Dredging in the model occurred once a year during the lowest stage at Acme. A cross section was dredged if the water depth anywhere in the designated 200-ft-wide navigation channel was less than 9 ft. A new-dredging routine was incorporated into HEC-6 to meet this specification. Two feet of overdredging was specified. Dredged material was removed from the river. During the 7-year simulation, approximately 4 million cu yd of material were dredged from the study reach downstream from Lock and Dam No. 1.

Trace Widths

22. Dredging requirements with 200-, 300-, 400- and 500-ft trace widths were compared. Dredging would be relatively insignificant with a 200-ft trace width. With a 300-ft trace width, most of the dredging requirements were met early (during the first 2 years) as existing crossings were removed. After this initial clearing, average annual dredging was estimated at 84,000 cu yd. Average annual dredging during the last 5 years was calculated to be 318,000 and 393,000 cu yd for the 400-ft and 500-ft trace widths, respectively. Compared to the base (no-dikes) condition, dredging was reduced in all the contracted channels except the 500-ft trace width. The slight increase in dredging with the 500-ft trace width, which is closest to the natural river width, is attributed to a decrease in sediment transport capacity caused by a decrease in channel width, which is not compensated for by an increase in velocity. Calculated dredging quantities are shown in Table 3, and total accumulated dredging is shown in Figure 11.

23. The effectiveness of the various trace widths in moving sediment through the study reach can be evaluated by comparing the sums of dredging and accumulated deposition. Accumulated deposition within the trace width can occur because only a 200-ft-wide navigation channel is dredged and because deposition in the navigation channel can occur below the authorized 9-ft

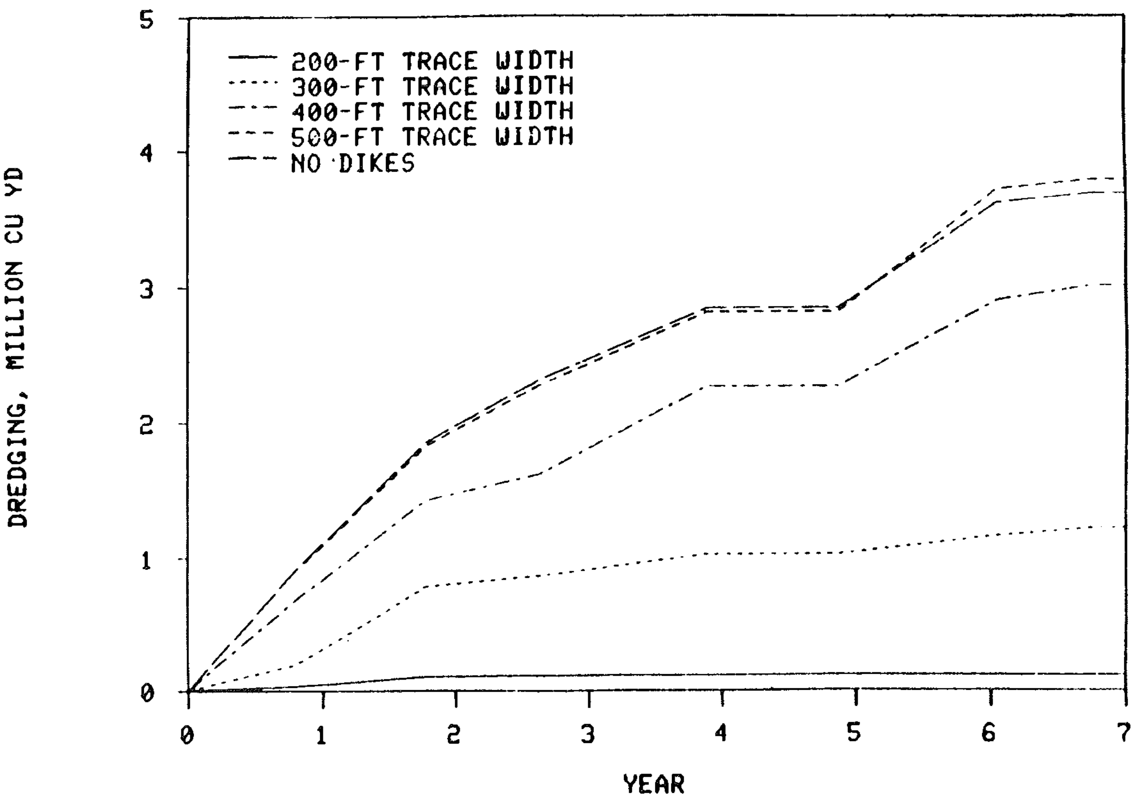


Figure 11. Accumulated dredging with tested trace widths, 1975-1981

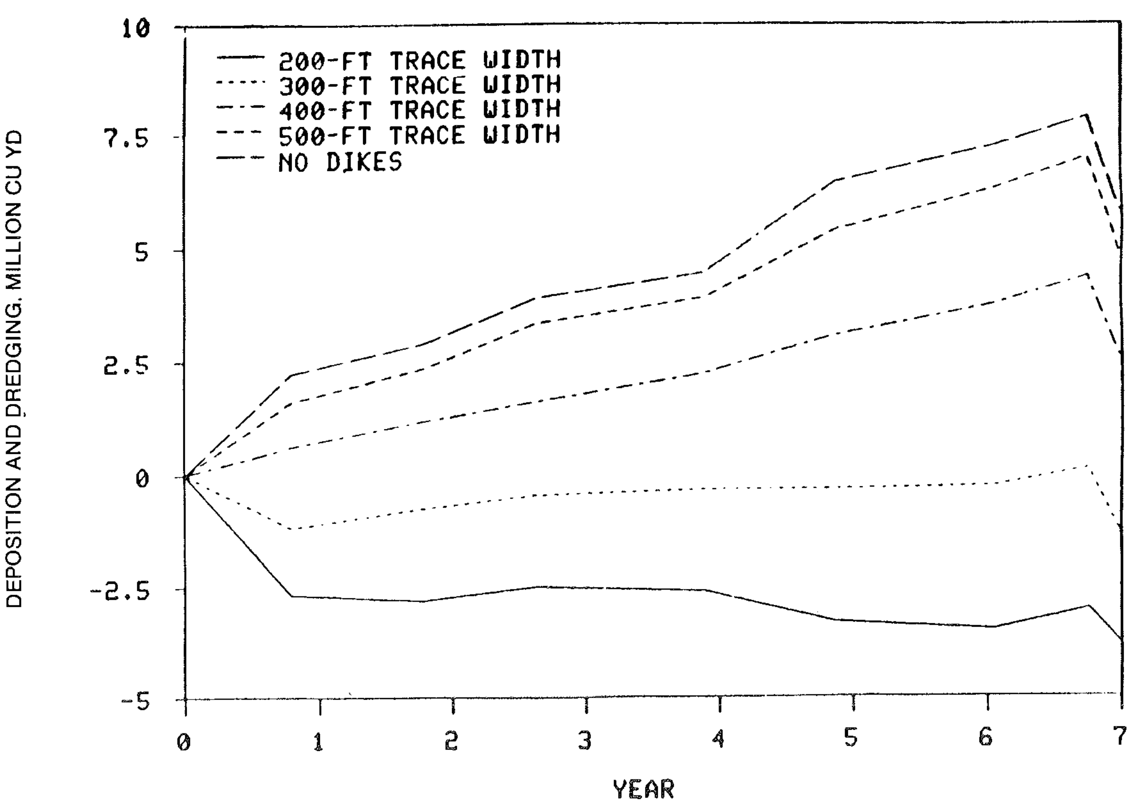


Figure 12. Accumulated dredging and deposition with tested trace widths, 1975-1981

depth. Dredging and accumulated deposition were calculated to be about 6 million cu yd in 7 years without constrictive works. Results with various trace widths are shown in Figure 12. With a 200-ft trace width, 3.8 million cu yd of material were removed from the study reach primarily because of scour. The effect of this scour on thalweg elevations is demonstrated in Figure 13. With a 300-ft trace width, deposition and dredging are essentially balanced and the thalweg profile is determined primarily by dredging requirements.

24. Contracting the river channel will generally result in an increase in velocity and depth. The effect of the trace widths on these hydraulic parameters was determined using the numerical model. Several discharges, ranging from 25,000 cfs to 142,000 cfs (navigation design flow), were tested. In these tests, starting water-surface elevations at Acme were assigned the same percent exceedance value as the discharge. (Stages and discharges were taken from Plates 22 and 4, USAED, New Orleans, 1980a.) Average channel velocity between Acme and Lock and Dam No. 1 was determined from the calculated channel velocities at 13 cross sections (Figure 14). At the navigation design flow, the 200-ft trace width increased average velocity over 100 percent to about 10 fps. The 300-ft trace width increased average velocity 60 percent to 7.6 fps. These increases may affect the navigability of the river. Changes in water-surface elevation with the constricted channel were relatively minor as shown in Table 4.

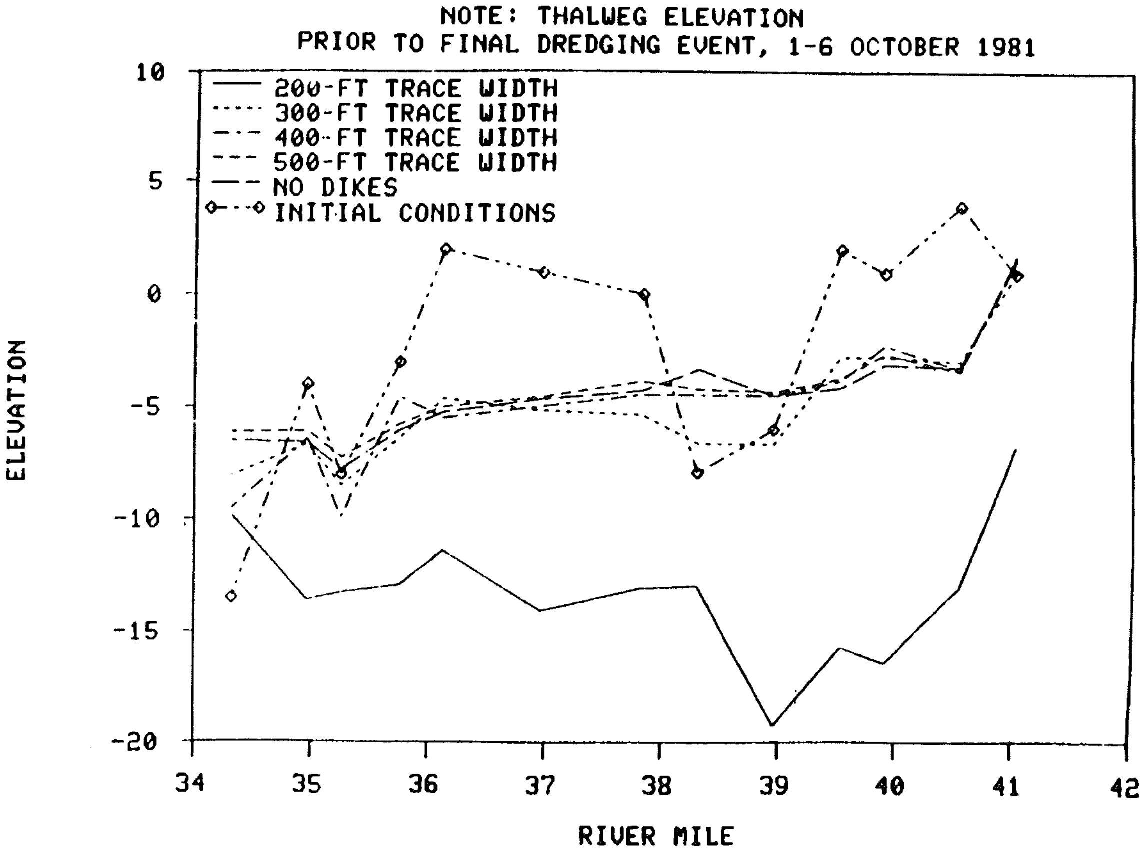


Figure 13. Calculated thalwegs for tested trace widths

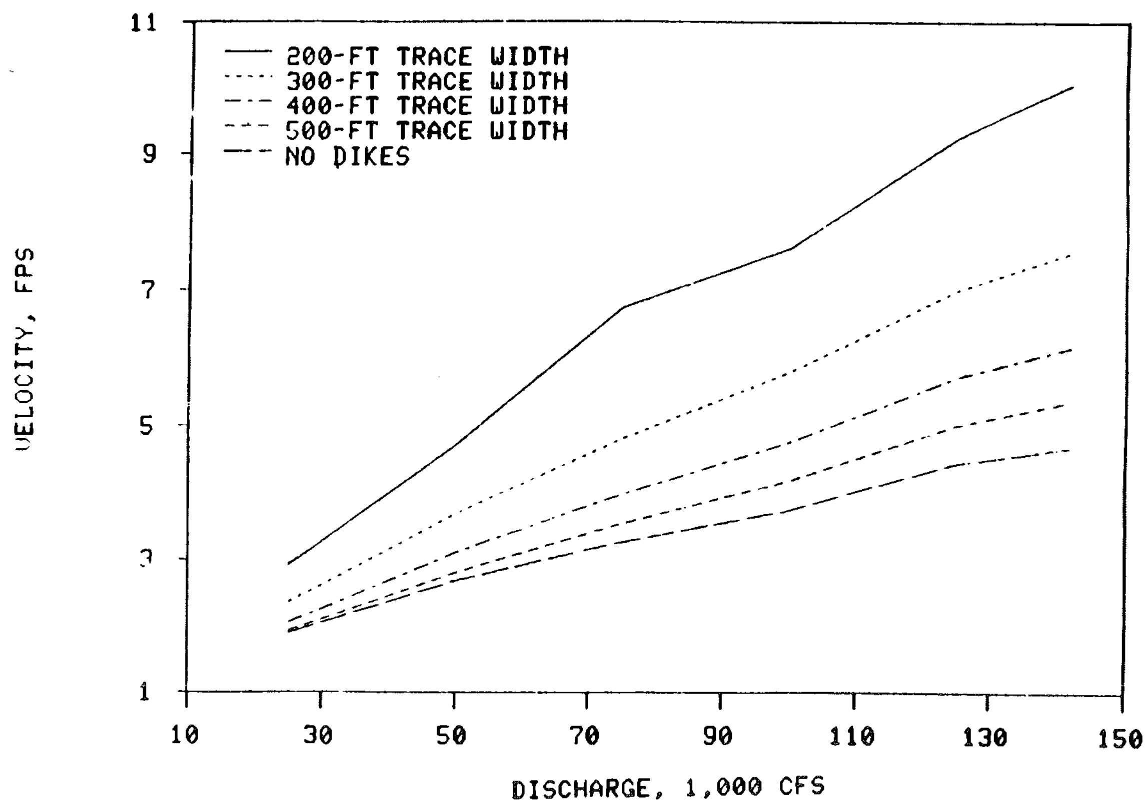


Figure 14. Calculated average channel velocities with tested trace widths, miles 34-41

PART V: 2-D MODEL

Description

25. The 2-D numerical model study was conducted using the TABS-2 modeling system (Thomas and McAnally 1985). This system, which consists of more than 40 computer programs to perform modeling and related tasks, provides 2-D solutions to open channel and sediment problems using finite element techniques. The major modeling components used in this study were RMA-2V and STUDH, which calculate 2-D depth-averaged flows and sedimentation, respectively. The other programs in the system perform digitizing, mesh generation, data management, graphical display, output analysis, and model interfacing tasks. Although TABS-2 may be used to model unsteady flow, in this study only steady-state conditions were simulated. Input data requirements for the hydrodynamic model (RMA-2V) include channel geometry, Manning's roughness coefficients, turbulent exchange coefficients, and boundary flow conditions. The sediment model (STUDH) requires hydraulic parameters from RMA-2V, sediment characteristics, inflow concentrations, and sediment diffusion coefficients. Sediment is represented by a single grain size, and transport is calculated using the Ackers-White equation (Ackers and White 1973). Due to the uncertainty related to the diffusion and exchange coefficients in the two models, prototype data for adjustment purposes are highly desirable.

Grid Generation

26. A finite element grid was developed to simulate about 1.5 miles of the Red River downstream from Lock and Dam No. 1 (Figure 15). The grid contained 931 elements and included the lock approach channel, I-wall, and

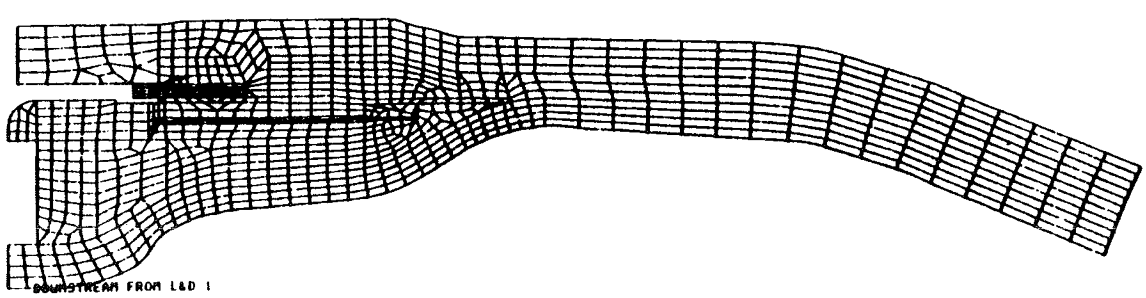


Figure 15. Finite element grid of reach downstream from Lock and Dam 1

exit channel. The floating guide wall was simulated by increasing the bottom elevation by 5.33 ft, which is the guide wall's submerged depth. Additional energy losses were accounted for by assigning a relatively high roughness coefficient (0.05). Initial bed elevations, which were obtained from construction drawings, represent conditions prior to opening of the structure. Slip boundaries were specified for most of the grid, allowing velocities to be calculated along the boundary. This method eliminates the need for an extremely fine grid adjacent to the boundary where the lateral velocity gradient is steep. Some of the boundary nodes were specified as stagnation points, i.e., locations of zero velocity. These specifications are generally located in corners of the grid and are employed to ease calculation of slopes for the slip boundaries. The tailwater at the downstream boundary and inflow at the upstream boundary were defined for each steady-state run. Boundary specifications used in the study are shown in Figure 16.

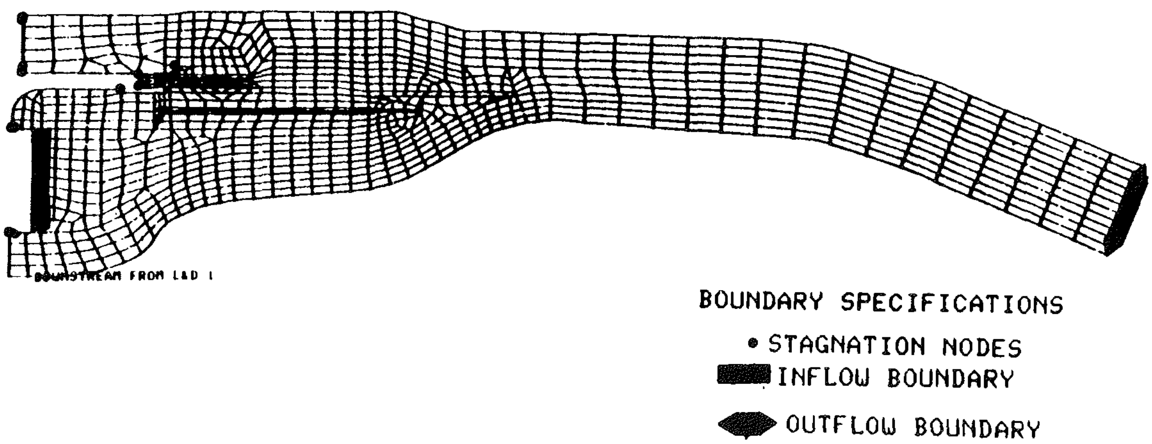


Figure 16. Boundary specifications

Hydrodynamic Boundary Conditions

27. The inflow distribution to the numerical model was based on the results of TABS-2 numerical model work upstream from Lock and Dam No. 1 conducted by USAED, Vicksburg (Little 1985). The calculated distribution percentage over the spillway for a discharge of 92,000 cfs was used for all flows in this study. The inflow velocity distribution is shown in Plate 1. These values were used to calculate unit discharges, which were then used as input for RMA-2V.

28. The model tailwater was determined using the backwater routine in HEC-6. Three cross sections were added to the previously developed 1-D model. Stages at Acme were used for the downstream tailwater, and the water-surface elevation calculated at river mile 42.65 was used as the tailwater for RMA-2V.

Roughness Coefficients

29. Manning's roughness coefficients were determined using predictive equations that include relative roughness as a variable and comparisons with measured data. The roughness coefficient for the channel sand bottom was set at 0.017. This value was used by the Vicksburg District in their study upstream from Lock and Dam No. 1 (Little 1985) and is based on grain size and water-surface elevation adjustment. Riprap placed on side slopes, downstream from the spillway, and in the lock approach channel has a D_{50} which varies between 24 and 36 in. The following equation (Anderson, Paintal, and Davenport 1970) defines n :

$$n = 0.0395\ D_{50}^{\ 0.1667}$$

where D_{50} is the grain size in feet of which 50 percent of the bed is finer. Roughness coefficients for the riprap vary between 0.039 and 0.041. The Limerinos equation (1970), an equation that considers the effect of relative roughness,

$$n = \frac{0.0926\ R^{\ 0.1667}}{1.16 + 2.0 \log \left(\frac{R}{D_{84}}\right)}$$

where R is the hydraulic radius in feet, yielded the same values for roughness coefficients when the depth exceeded 20 ft. The roughness coefficient for the channel riprap bottom was set at 0.040. The Limerinos equation increased the roughness coefficient to 0.045 at a depth of 10 ft and to 0.085 at a depth of 1 ft. These results were found to be similar to calculations using equations by Chow (1959) and Leopold, Wolman, and Miller (1964). Calculated velocity distributions, using roughness coefficients within the range of predicted values, were compared to measured velocity distributions taken in the exit channel about one-half mile downstream from the spillway. The channel cross section was modified in the numerical model to account for bank erosion (Figure 17). Separate roughness coefficients were assigned to the side slopes and the boundary elements. To maintain numerical stability, boundary elements were designed to be submerged in the numerical model and therefore require a somewhat greater roughness value to balance conveyance. Roughness coefficients of 0.040 for the side slopes and 0.050 for the boundary elements were found to reproduce the measured velocity distributions adequately (Plate 2).

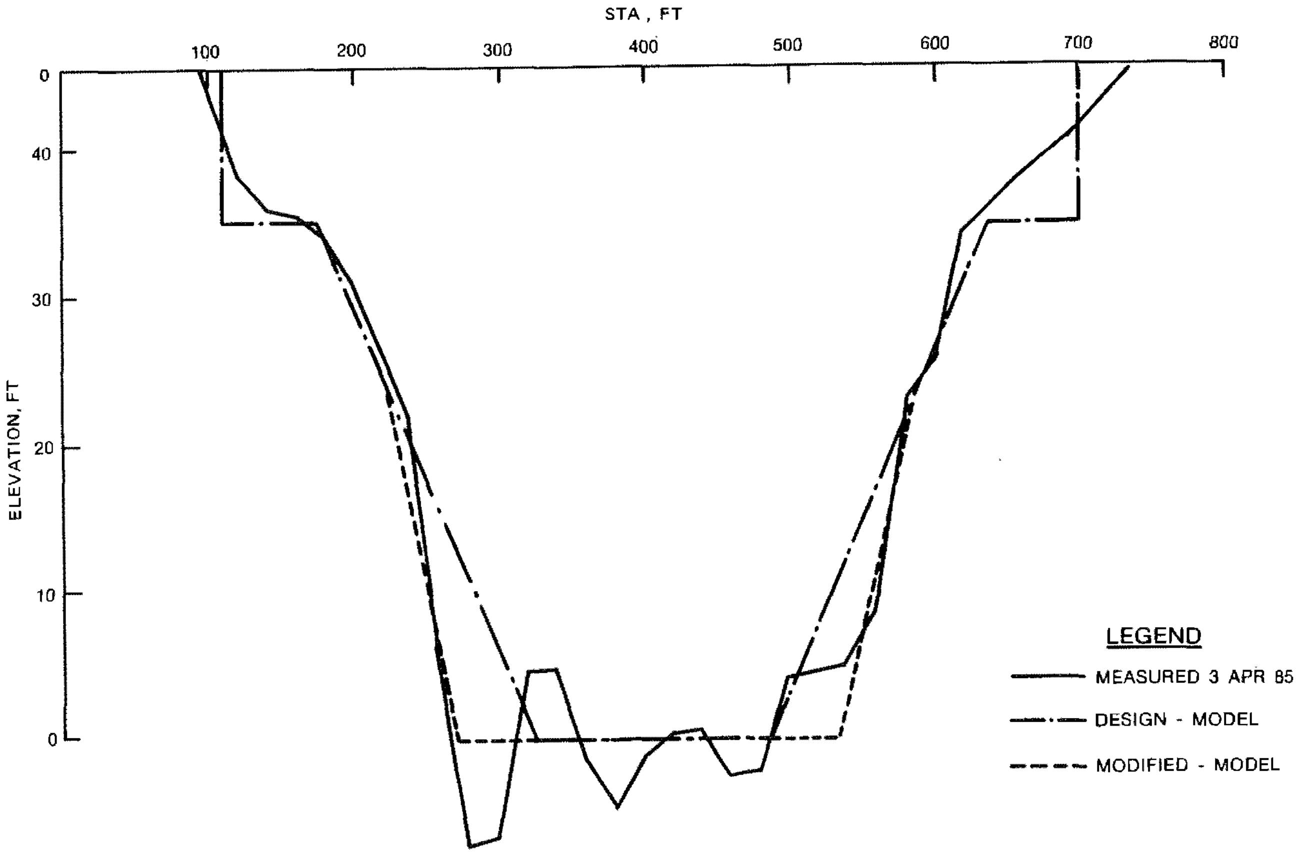


Figure 17. Exit channel cross section

Turbulent Exchange Coefficients

30. Momentum exchanges due to velocity gradients are approximated by multiplying a turbulent exchange coefficient times the second derivative of the velocity with respect to the x and y directions, Exx and Eyx ; and Exy and Eyy , respectively. A sensitivity study was conducted to test the

influence of the turbulent exchange coefficients on flow patterns in the numerical model. Coefficients between 10 and 50 $lb\text{-}sec/ft^2$ were tested with a discharge of 92,000 cfs and a tailwater of 44.5 ft. The magnitude of the exchange coefficients did not appear to have any significant effect on general flow patterns (Plates 3-6). In all cases, an eddy developed on the left descending bank of the lock approach channel. The strength of the eddy into the lock chamber itself is compared in Plates 7-11. The strongest eddy occurred with coefficients of 15, but differences were small. Similar flow patterns were observed in the prototype, but there were no measurements to compare with calculated results.

31. The effect of the turbulent exchange coefficient on velocity distribution in the exit channel was also tested. Velocities calculated at a discharge of 92,000 cfs and a tailwater of 44.5 ft with coefficients ranging between 5 and 50 $lb\text{-}sec/ft^2$ were compared with measured velocities (Plate 12). There is more flow in the center of the channel with the lower exchange coefficients, but all of the calculated velocity distributions fit within the measured data scatter.

32. A turbulent exchange coefficient of 25 $lb\text{-}sec/ft^2$ has provided satisfactory results in previous numerical model investigations. The Vicksburg District determined that this value produced reasonable flow patterns in the numerical model study upstream from Lock and Dam No. 1. The numerical solution converges fairly well with a coefficient of 25. Convergence is more difficult with lower exchange coefficients. Due to the apparent lack of sensitivity of the hydrodynamics to the turbulent exchange coefficients tested, turbulent exchange coefficients of 25 $lb\text{-}sec/ft^2$ were used in this study.

Bed Material

33. The TABS-2 system analyzes sediment movement using a representative grain size. This technique works well when the bed material is fairly uniform. Unfortunately, bed material size varies considerably around the structure and laterally across the channel. Upstream from Lock and Dam No. 1 at river mile 51.5, the median bed material size varied between 0.13 mm at the point bar and 0.65 mm at the thalweg during measurements taken in April and May 1985. Bed samples from deposits in the upstream and downstream lock

approach channels had D_{50} values between 0.07 and 0.04 mm. Inside the lock chamber itself, a sample near the upstream gate had a median size of 0.028 mm, and a sample near the downstream gate had a median size of 0.055 mm. In addition, bed material size was observed to vary with depth in the deposit. This variation with depth is due to layering of different sizes of fairly uniform material and is attributed to changes in flow patterns at different discharges and stages. Since the complex variation of bed material size cannot be accounted for in the numerical model, a representative size must be selected for the area of primary interest. An approximation of the bed material size was sufficient, because appropriate response in the numerical model was adjusted using the sediment diffusion coefficients.

34. Based on a sample taken from the deposit in the upstream lock approach channel in November 1984, the Vicksburg District chose an average grain size of 0.07 mm for its numerical model study upstream from Lock and Dam No. 1. Material taken from a bucket dredge just downstream from the downstream miter gate in March 1985 was slightly finer with a D_{50} of 0.065 mm. The total deposit depth was about 30 ft at this location. Gradation curves for these samples are shown in Plate 13. Differences between the upstream and downstream sample gradations were not considered great enough to require different values in the two numerical models; therefore, for the sake of consistency, an average grain size of 0.07 mm was used in the numerical model study downstream from Lock and Dam No. 1.

Sediment Concentration

35. The sediment inflow concentration for the numerical model is a function of the representative grain size used in the study. Only the portion of the total sediment load that contributes to bed changes in the primary area of interest is included. Based on a bed material gradation with a median diameter of 0.07 mm in the upstream lock approach channel, it was determined that material greater than 0.03 mm would be considered in determining sediment inflow. This includes 90 percent of the material found in the bed. The remaining 10 percent can be considered wash load (Einstein 1950).

36. Since most of the bed material was very fine sand and very coarse silt (0.125-0.031 mm), suspended sediment data with size class breakdowns in both the sand and silt ranges were required to determine the appropriate

sediment inflow concentration. The Vicksburg District made suspended sediment measurements upstream from Lock and Dam No. 1 at river mile 51.5 in April and May 1985. At a discharge of 59,500 cfs, a total suspended sediment concentration of 771 mg/ℓ was measured; and at 93,000 cfs, the concentration was 1,525 mg/ℓ. These concentrations are compared with concentrations at Alexandria for 1971, 1972, and 1975-1981, in Figure 18. The 1985 concentrations are

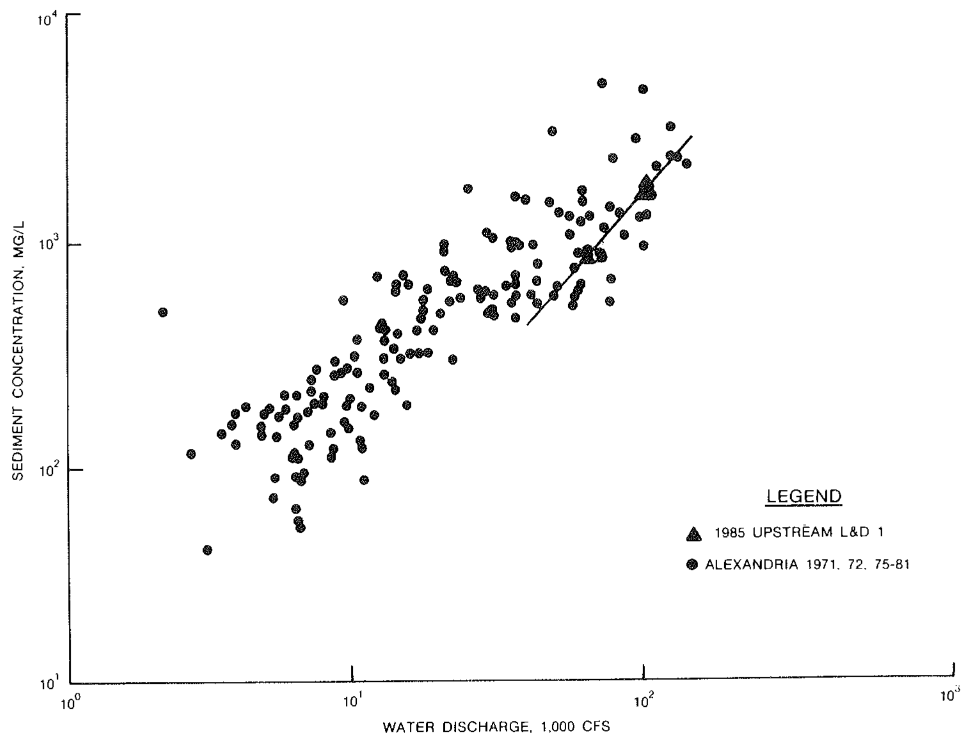


Figure 18. Comparison of measured suspended sediment concentration at Alexandria and upstream from Lock and Dam No. 1

within the range of data, but at the lower end. This distribution may have occurred because the 1985 data were taken well into the runoff season when concentrations typically decline. The 1985 data did have size class analyses in the silt range so that appropriate sediment concentrations could be determined. An additional 20 percent reduction in the measured sediment concentration was made to account for the calculated sediment concentration reduction between the measurement site and the dam determined by the Vicksburg District in their upstream model study. Extrapolation and interpolation from these two data points were used to determine sediment inflow concentration at the upstream boundary of the model for various discharges (Figure 19).

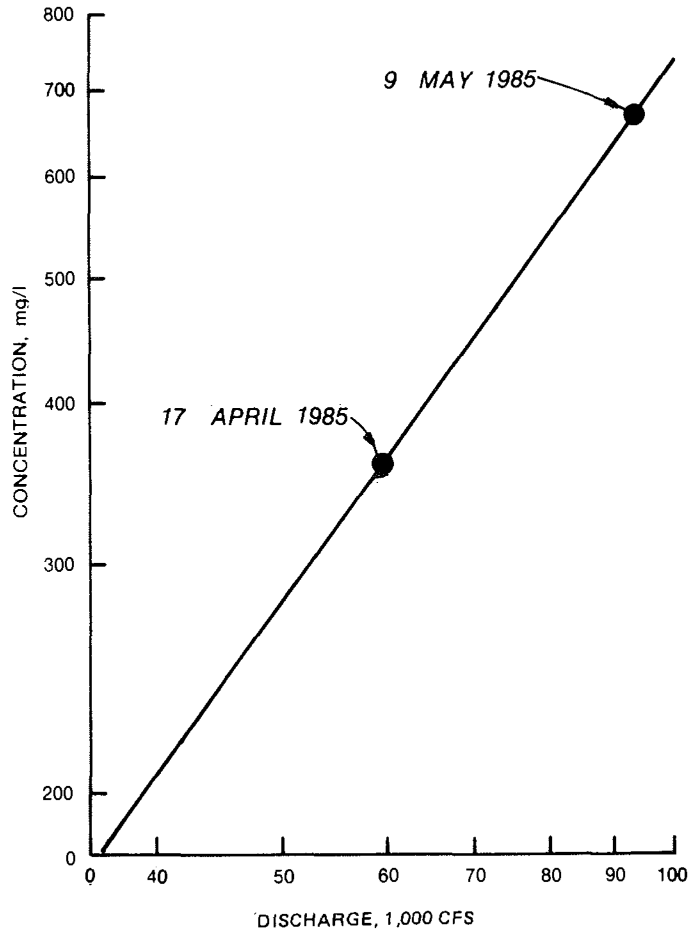


Figure 19. Sediment inflow rating curve for D_{50} of 0.07 mm

37. Steady-state or equilibrium sediment concentrations are calculated at each node during a spinup run with the STUDH program. These calculations can be accomplished within a few time-steps at a constant discharge. These equilibrium concentrations are then used as initial conditions in the normal STUDH run. This procedure eliminates the problem of assuming a constant initial concentration for the entire grid, which results in rapid deposition in slack-water areas.

Sediment Diffusion Coefficients

38. A sensitivity study was conducted to determine the influence of the sediment diffusion coefficients on sediment deposition in the numerical model. Deposition was calculated for a 5-day period with a discharge of 92,000 cfs

and an inflowing sediment concentration of 670 mg/ℓ. Initial concentrations throughout the grid were calculated using spinup runs. Sediment diffusion coefficients between 25 and 0.1 m^2/sec were calculated. Sediment deposition during the 5-day simulation at four locations in the grid is compared in Table 5. In the lock approach and spillway exit channels, the calculated sediment deposition does not appear to be sensitive to the magnitude of the sediment diffusion coefficients. In these areas, flow is moving generally in a downstream direction and conveyance is the primary driving force affecting sediment movement. However, at the lock gate, the magnitude of the sediment deposition is significantly influenced by the sediment diffusion coefficients. The lock gate is essentially in a dead-water area where sediment deposition is primarily a function of diffusion. The sensitivity study demonstrated the critical importance of the coefficients for predicting deposition in the vicinity of the lock gate.

39. Hydrographic survey data in the downstream lock approach channel taken on 1 May 1985 were used to adjust the numerical model. Prior to this survey, about 52,000 cu yd were dredged between the end of the floating guide wall and 400 ft downstream. This amount represents about 17 percent of the material deposited in the approach channel. Deposition recorded by this survey includes the sediment accumulated since the structure was opened just prior to the 1984-1985 runoff season.

40. Boundary conditions for an October 1984-May 1985 simulation period were determined for the numerical model. A discharge histograph* was developed from daily discharge measurements at Alexandria and Lock and Dam No. 1 (Plate 14). Daily discharge measurements at Alexandria were used prior to 12 December 1984, when daily measurements at Lock and Dam No. 1 were started. A stage histograph was developed from daily stage measurements downstream from Lock and Dam No. 1 supplemented by daily stage records at Acme (Plate 15). The Acme record was adjusted for head losses between the two gages and used to fill gaps in the Lock and Dam No. 1 record. Sediment concentrations at the upstream boundary were assigned from the rating curve developed from suspended sediment measurements taken upstream from Lock and Dam No. 1 (river mile 51.3) in April and May 1985 (Figure 19). Input for the histograph simulation is listed in Table 6.

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* A hydrograph simulated by a series of steady-state events of varying durations is called a histograph.

41. A preliminary sediment diffusion coefficient was selected using an approximate technique for simulating deposition between October 1984 and May 1985. Deposition at selected elements in the lock approach channel was calculated for a high-water condition (92,000 cfs, stage = 44.5 ft) and a low-water condition (78,000 cfs, stage = 36.2 ft). Histograph events were categorized as either high- or low-water events depending on whether their stage exceeded the top of the I-wall (el 38). The product of duration, discharge, and inflow concentration for each histograph event was divided by the same product for either the high- or low-water base condition to establish a correction factor. This factor was multiplied by the deposition that occurred during the base condition to determine an estimated deposition for that histograph event. Total deposition for the period was calculated by adding deposition from each event.

42. The approximate technique was used to estimate sediment deposition with sediment diffusion coefficients of 5 and 2. With a coefficient of 5, calculated sediment deposition between the lock gate and the end of the floating guide wall (800 ft) was 125 percent of the measured deposition; and between the lock gate and the end of the lock approach channel (1,800 ft), 84 percent of the measured deposition. With a coefficient of 2, calculated sediment deposition between the lock gate and the end of the floating guide wall was 104 percent of the measured deposition; to the end of the approach channel, 79 percent of the measured deposition was calculated. A sediment diffusion coefficient of 2 was selected for a detailed simulation.

43. The numerical model was used to make a detailed simulation of deposition and scour in the lock approach channel. Hydrodynamic calculations were made for each steady-state event in the October 1984 to May 1985 histograph. Channel geometry was updated at the end of each event to account for scour or deposition. Steady-state sediment concentrations were calculated and used as initial conditions before each sediment run. Results of the simulation are shown in Figure 20. The simulation was especially good for the first 500 ft downstream from the lock gate. For the next 1,000 ft, the model predicted about 75 percent of the measured deposition. The numerical model was deemed to have successfully simulated the prototype in the primary area of interest—downstream from the lock gate—and could be used to evaluate design alternatives.

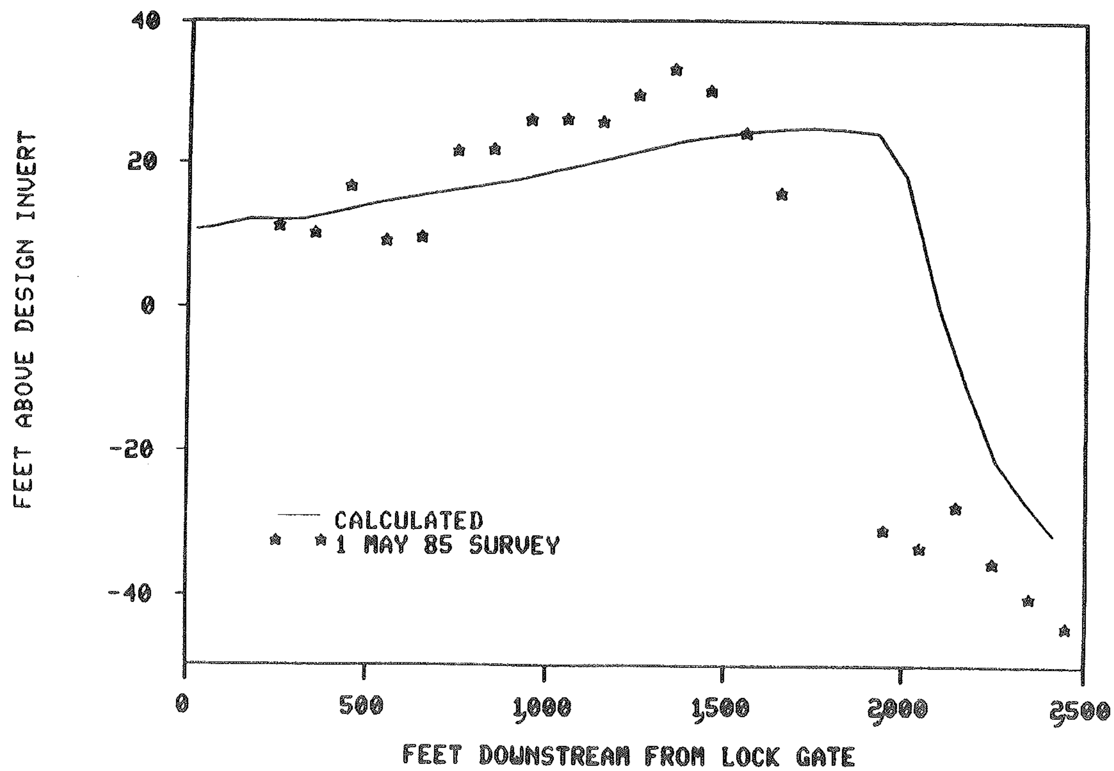

Figure 20. Comparison of measured and calculated deposition in approach channel

PART VI: MODEL RESULTS

Initial Design Alternatives

44. Two design changes were initially proposed to decrease the strength of the eddy, and thereby the quantity of sediment, into the lock chamber. Alternative 1 consisted of raising the I-wall to el 60 for 300 ft. This alternative would provide a barrier between the lock approach and spillway exit channels for a total of 450 ft downstream from the lock gate. Alternative 2 included the 300-ft-long extension of the I-wall and a 250-ft-long longitudinal dike on the left descending bank downstream from the lock wing wall (Figure 21).

45. Alternatives 1 and 2 were tested with a discharge of 92,000 cfs and a tailwater of 44.5 ft. Velocity vectors are compared with the existing conditions in Plates 16-18. The 300-ft-long extension reduced the strength of

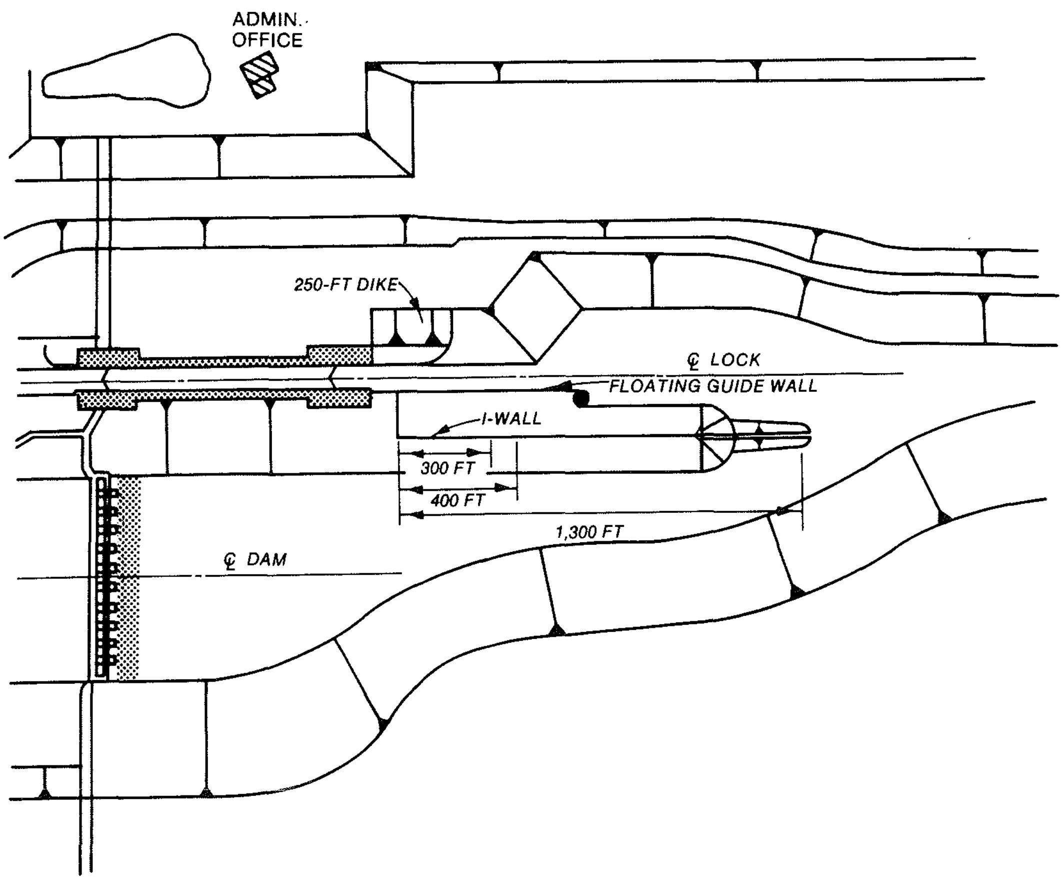

Figure 21. Alternative designs

the eddy into the lock chamber. There was no apparent additional benefit from the dike.

46. The effect of Alternatives 1 and 2 on sediment deposition at the lock gate was tested with a discharge of 92,000 cfs, a tailwater of 44.5 ft, an inflowing sediment concentration of 670 mg/ℓ, and a duration of 10 days. A comparison of Alternatives 1 and 2 with existing conditions showed that a 45 percent reduction was achieved with both alternatives. With no apparent additional benefit from the dike, Alternative 2 was dropped from consideration.

Additional Raising of the I-wall

47. Alternatives 3 and 4 called for raising the I-wall for a total of 400 ft and 1,300 ft, respectively (Figure 21). With Alternative 4 the entire I-wall would be raised to el 60. Using the same flow conditions in the numerical model as before, it was determined that Alternative 3 would provide a 72 percent reduction in existing deposition at the lock gate and Alternative 4 would provide a 97 percent reduction. A deposition profile in the lock approach channel for the simulated period is shown in Figure 22. This figure

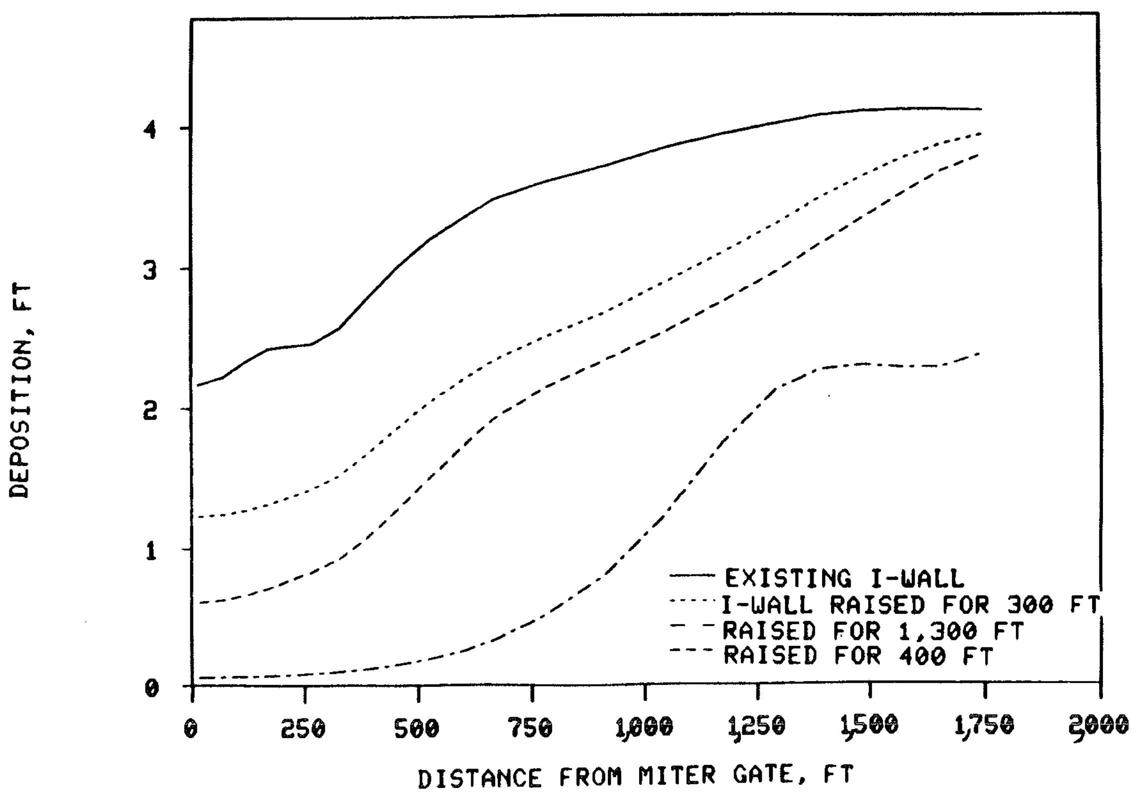

Figure 22. Effect of raising I-wall on deposition in back approach channel

demonstrates that the percentage reduction in deposition at the lock gate is not representative of the rest of the approach channel. There will still be significant deposition in the lock approach channel with any of the alternatives tested. Deposition contour maps are shown in Plates 19-21.

PART VII: CONCLUSIONS AND RECOMMENDATIONS

48. The results of the 1-D numerical model study can be used to evaluate alternative trace widths for the Red River downstream from Lock and Dam No. 1. Calculated quantities of dredging depend on the sediment inflow, bed material gradation assumptions, hydrograph characteristics, and the assumption that all flow is confined to the trace width being tested. The 1-D numerical model does not produce the pool crossing sequence nor does it include deposition of fine material that would be expected between dikes or in other slackwater areas. For these reasons, calculated dredging and deposition quantities should be considered relative and not as quantitative estimates or long-term average annual values. Model study results suggest that excessive scour would occur with a 200-ft trace width, that a 500-ft trace width would have no significant effect, and in terms of maintenance, that a 300-ft trace width would be appropriate.

49. This study has evaluated the effectiveness of various trace widths on sediment movement through the study reach. It is also necessary to consider the possible effect of the contraction works on design flood flows and navigability. This question could be addressed in a more detailed study that incorporates the following: (a) better definition of the dikes, including the top elevation and sloping dike faces; (b) inclusion of areas between dikes accounting for deposition and increase of roughness due to vegetation; and (c) inclusion of overbank areas for flood flow conveyance. To define long-term dredging quantities more accurately, a long-term hydrograph, i.e., 50 years, should be run with extreme events including hypothetical floods included.

50. The 2-D model study demonstrated that significant deposition problems in the downstream lock approach channel will continue with the existing design. Deposition against the downstream lock miter gate can be reduced by raising the I-wall above the water level. Deposition at the gate will decrease as the distance between the end of the raised I-wall and the lock gate increases.

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Table 1

Average Annual Bed Material Gradation, Alexandria

	Percent in Size Class					
Year	Silt	Very Fine Sand	Fine Sand	Medium Sand	Coarse Sand	Very Coarse Sand
1971	1.6	15.9	59.9	18.8	3.4	0.2
1972	8.2	46.3	29.6	13.8	2.0	0.1
1975	8.9	44.4	38.1	8.0	0.5	0.1
1976	26.0	47.7	20.6	4.6	1.0	0.2
1977	34.5	32.0	16.9	15.7	0.7	0.2
1978	17.8	43.0	35.4	2.9	0.6	0.1
1979	10.2	42.8	30.2	12.7	3.0	0.6
1980	3.0	39.7	48.0	8.8	0.4	0.1
1981	2.8	24.2	57.4	15.5	0.1	--
Average	12.6	37.3	37.3	11.2	1.3	0.2

Table 2

Manning's Roughness Coefficients

	Discharge in 1,000 cfs				
River Mile	200	125	90	30	4
34.31	0.017	0.017	0.021	0.021	0.022
69.02	0.021	0.023	0.024	0.024	0.024
103.02	0.020	0.021	0.023	0.029	0.031
127.03	0.024	0.026	0.027	0.030	0.032
156.81	0.025	0.025	0.026	0.032	0.032
181.99	0.015	0.016	0.016	0.024	0.032

Table 3

Calculated Dredging Quantities

	Dredging, 1,000 cu yd				
	No	Trace Widths, ft			
Hydrograph	Contraction	500	400	300	200
1975	900	904	677	193	28
1976	954	911	739	590	71
1977	457	455	195	78	9
1978	529	544	649	159	--
1979	--	--	--	--	--
1980	770	900	634	131	--
1981	67	68	110	54	--
Total	3,667	3,782	3,004	1,205	108

Table 4

Water-Surface Elevation at River Mile 41

Discharge	No	Trace Width, ft			
cfs	Contraction	200	300	400	500
25,000	29.8	29.9	29.9	29.8	29.8
50,000	40.0	40.3	40.1	40.0	40.0
75,000	46.6	47.2	46.8	46.6	46.6
100,000	52.1	52.9	52.5	52.2	52.1
125,000	54.1	54.9	54.5	54.2	54.1
142,000	57.1	58.1	57.6	57.3	57.1

Table 5

Sensitivity of Deposition to Sediment Diffusion Coefficients

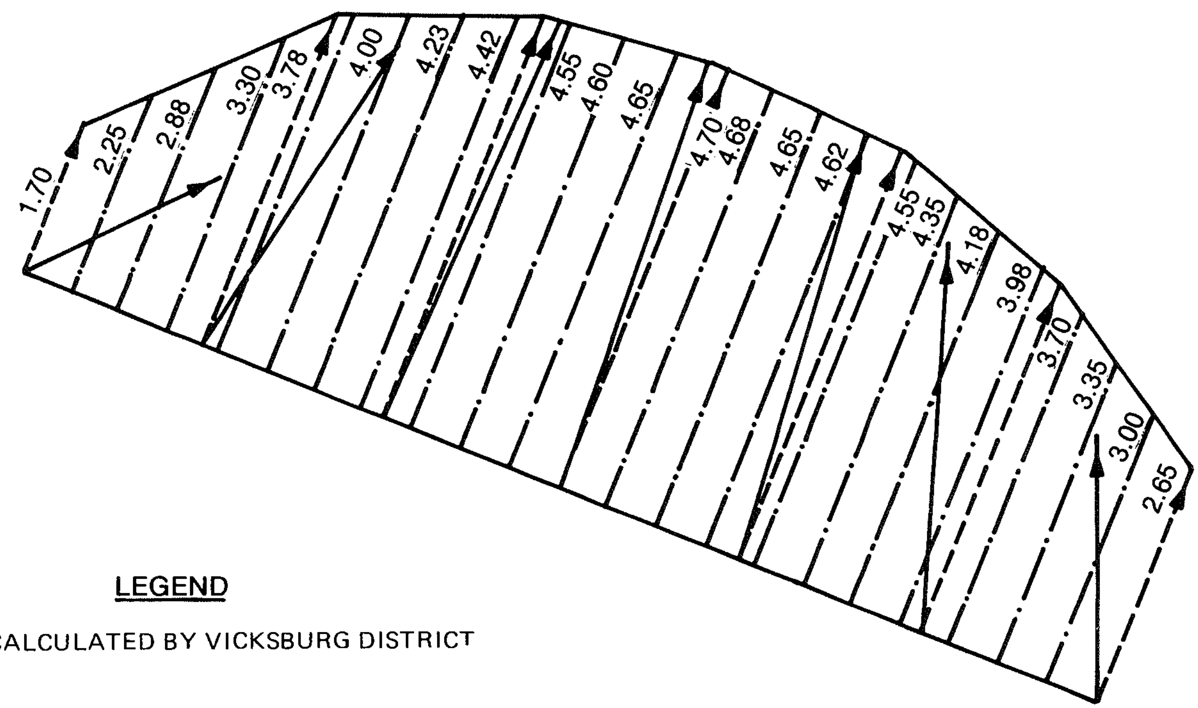
Sediment Diffusion Coefficient m^2/sec	Deposition, ft: Lock Gate	Lock Approach Channel 1,000 ft Downstream from Gate	Lock Approach Channel 1,400 ft Downstream from Gate	Spillway Exit Channel 600 ft Downstream from Dam
25.0	1.9	1.9	2.0	2.3
15.0	1.9	1.9	1.9	2.3
5.0	1.5	1.9	2.0	2.3
2.0	1.1	1.8	2.0	2.4
0.5	0.3	1.9	2.2	2.4
0.1	0.1	2.1	2.2	2.4

Note: Original (Type 1) design; discharge = 92,000 cfs for 5 days; sediment inflow concentration = 670 mg/ℓ; representative grain size = 0.07 mm.

Table 6

Adjustment Simulation Histograph

Simulation No.	Period: Start	Period: Finish	Time days	Discharge cfs	Tailwater ft	Inflow Concentration mg/ℓ
1	20 Oct	26 Oct	7	42,000	28.0	220
2	27 Oct	04 Nov	9	71,000	37.0	460
3	05 Nov	08 Nov	4	82,000	39.0	560
4	09 Nov	12 Dec	34	58,000	37.0	350
5	13 Dec	21 Dec	9	39,000	31.5	200
6	22 Dec	30 Dec	9	74,000	37.0	470
7	31 Dec	06 Jan	7	57,000	37.0	370
8	07 Jan	15 Jan	9	68,000	40.0	430
9	16 Jan	23 Jan	8	38,000	37.5	190
10	24 Jan	11 Feb	19	38,000	28.0	190
11	12 Feb	01 Mar	18	48,000	35.0	280
12	02 Mar	13 Mar	12	92,000	44.5	670
13	14 Mar	25 Mar	12	62,000	44.5	380
14	26 Mar	14 Apr	21	92,000	44.5	670
15	15 Apr	30 Apr	15	43,000	39.5	230



LEGEND

CALCULATED BY VICKSBURG DISTRICT

PROJECTED PERPENDICULAR TO AXIS

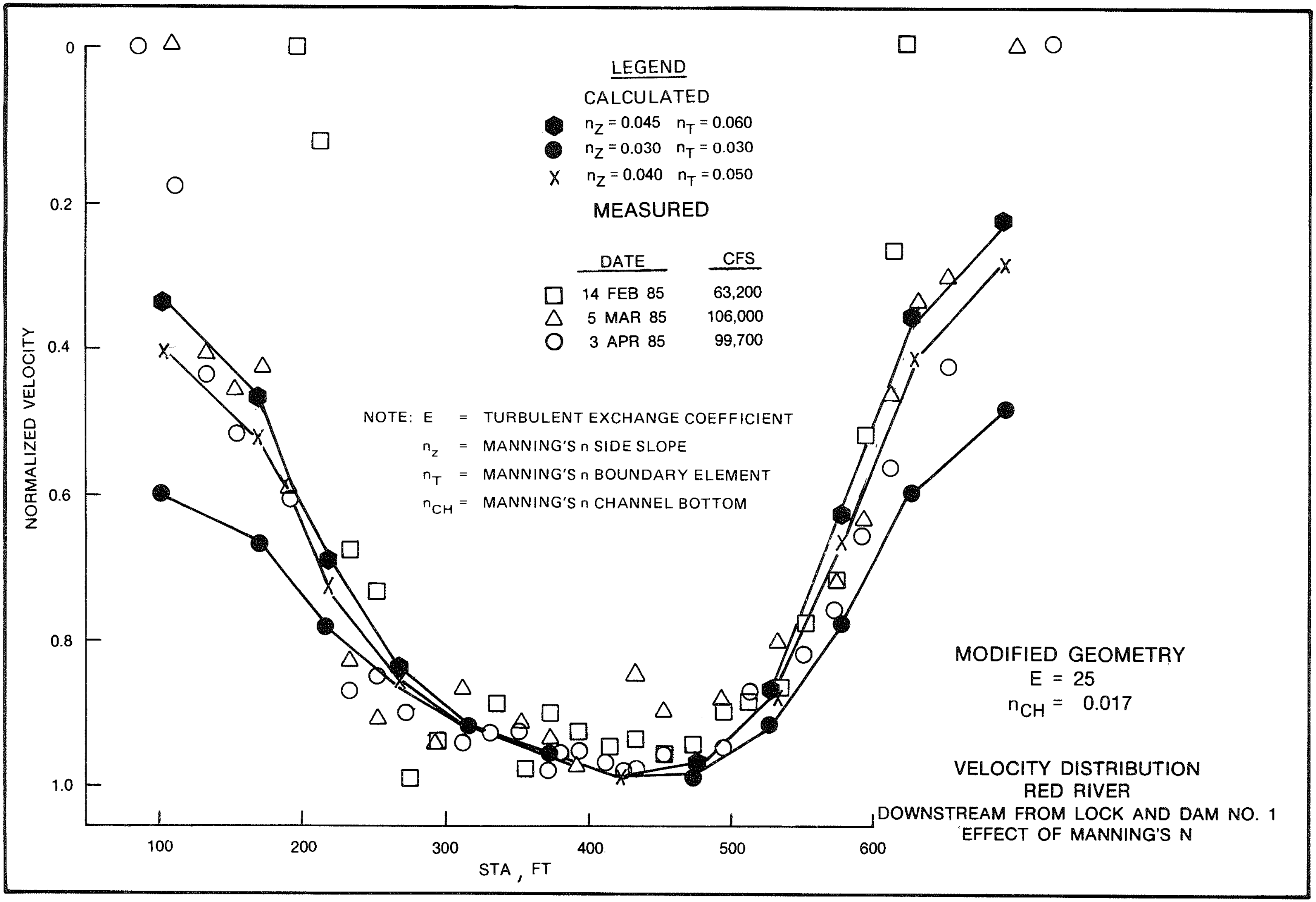
RELATIVE VECTORS AT GRID NODES, fps

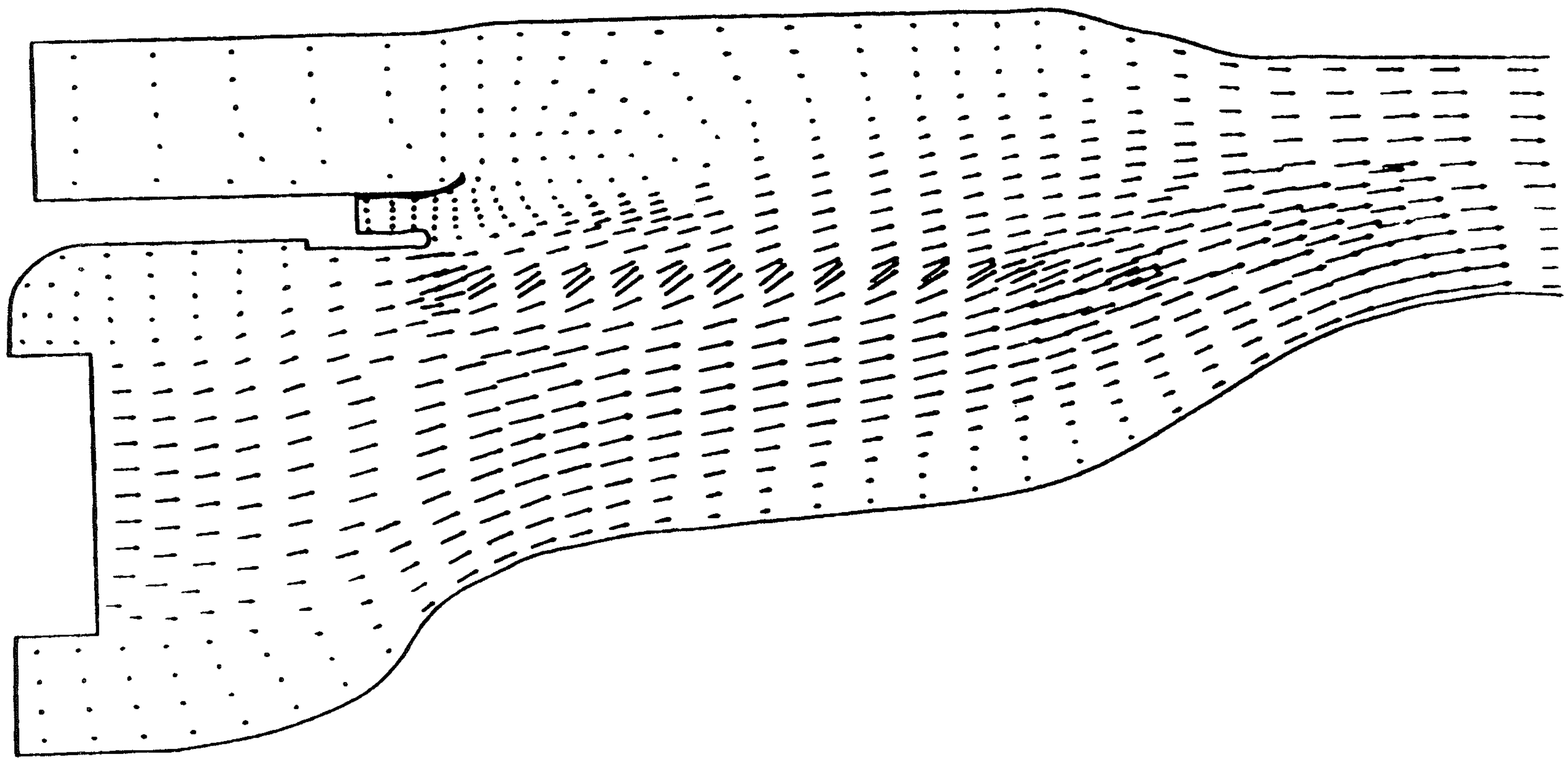
INFLOW VELOCITY DISTRIBUTION

Q = 92,000 cfs

TAILWATER ELEVATION = 46.6

PLATE 2


LEGEND
CALCULATED
n_Z = 0.045 n_T = 0.060
n_Z = 0.030 n_T = 0.030
X n_Z = 0.040 n_T = 0.050
MEASURED
DATE CFS
14 FEB 85 63,200
5 MAR 85 106,000
3 APR 85 99,700
NOTE: E = TURBULENT EXCHANGE COEFFICIENT
n_z = MANNING'S n SIDE SLOPE
n_T = MANNING'S n BOUNDARY ELEMENT
n_{CH} = MANNING'S n CHANNEL BOTTOM
NORMALIZED VELOCITY
0
0.2
0.4
0.6
0.8
1.0
100
200
300
400
500
600
STA, FT
MODIFIED GEOMETRY
E = 25
n_{CH} = 0.017
VELOCITY DISTRIBUTION
RED RIVER
DOWNSTREAM FROM LOCK AND DAM NO. 1
EFFECT OF MANNING'S N



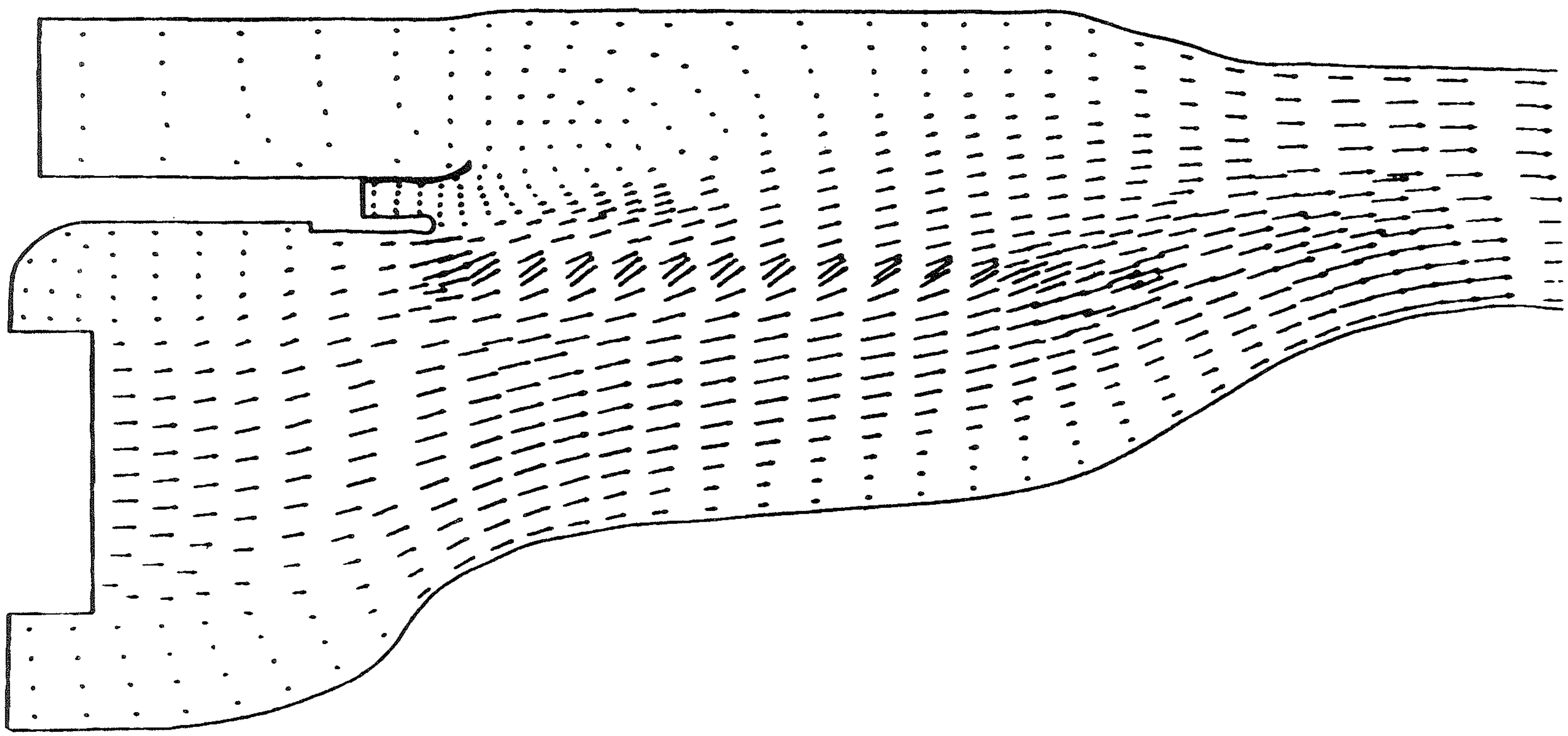
DOWNSTREAM LOCK AND DAM NO. 1

ORIGINAL DESIGN

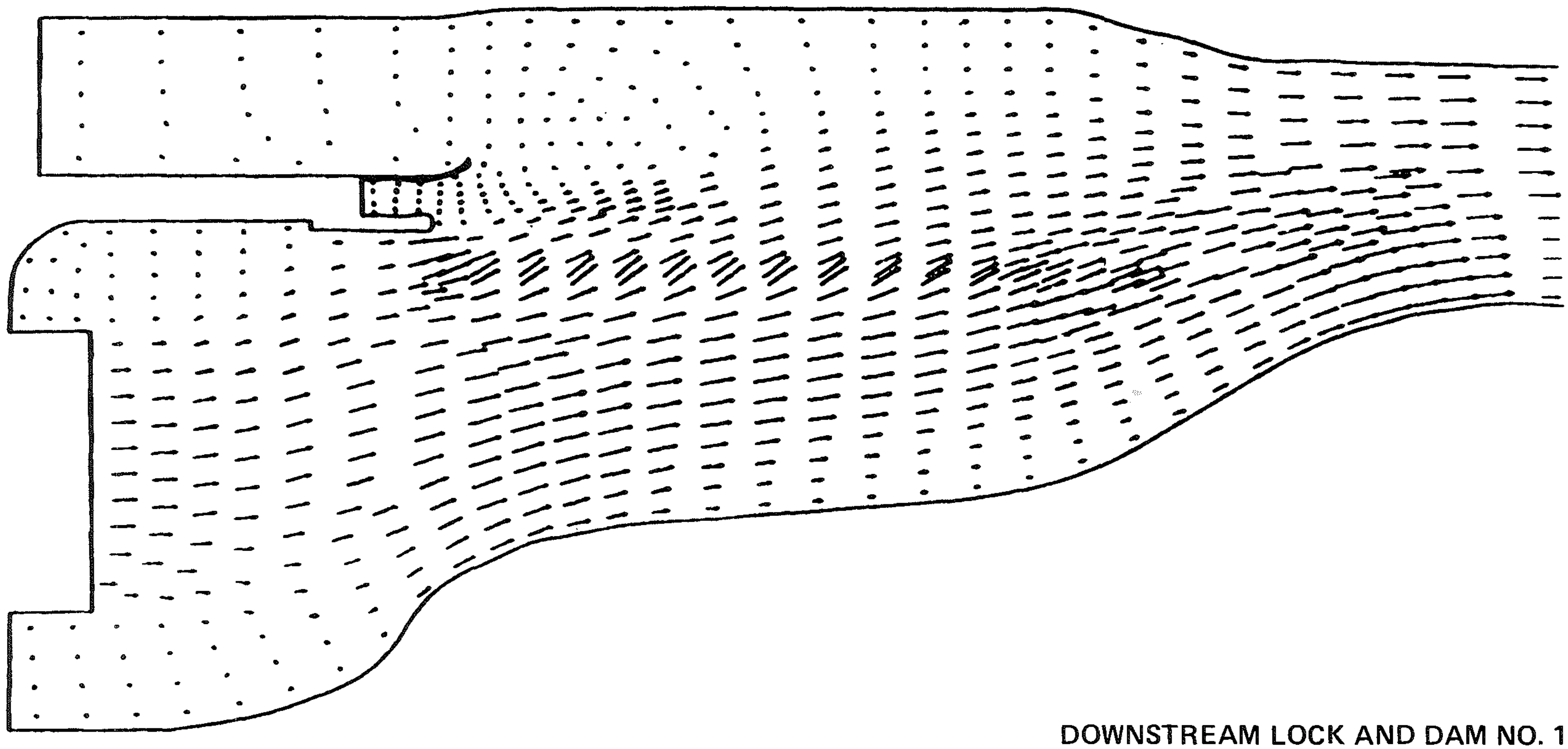
VELOCITY PATTERNS

E = 10

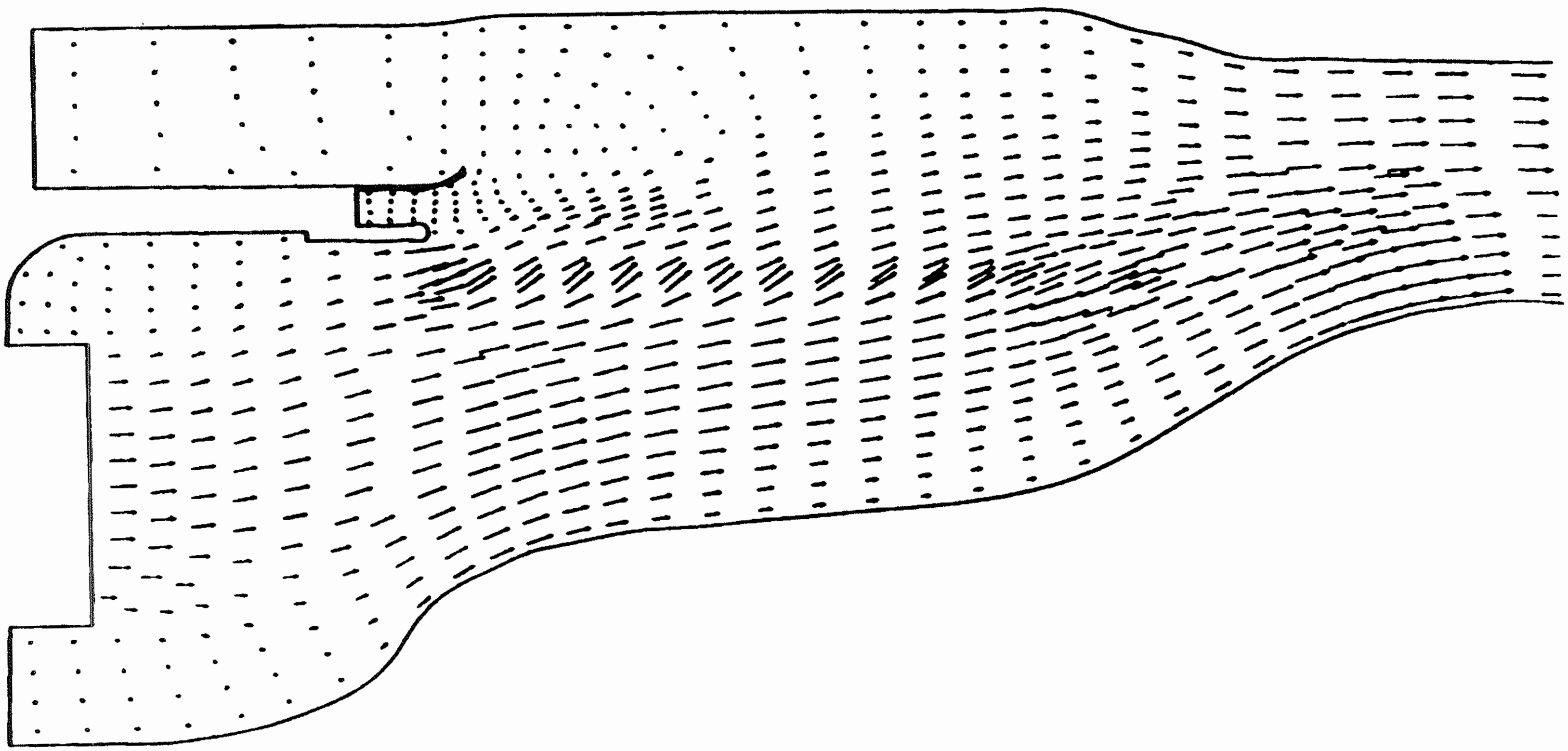
PLATE 4



DOWNSTREAM LOCK AND DAM NO. 1
ORIGINAL DESIGN
VELOCITY PATTERNS
E = 15



DOWNSTREAM LOCK AND DAM NO. 1
ORIGINAL DESIGN
VELOCITY PATTERNS
E = 25

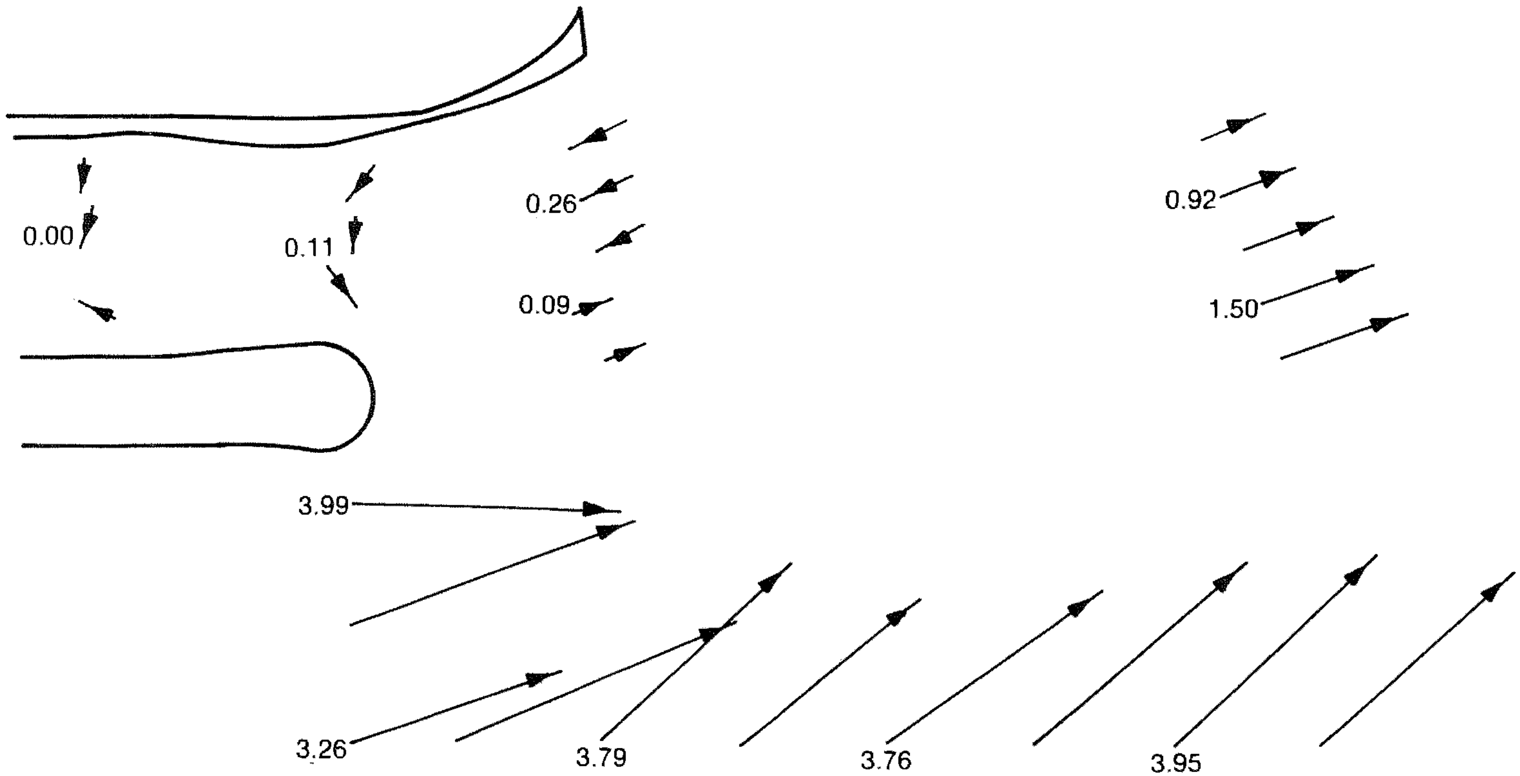


DOWNSTREAM LOCK AND DAM NO. 1

ORIGINAL DESIGN

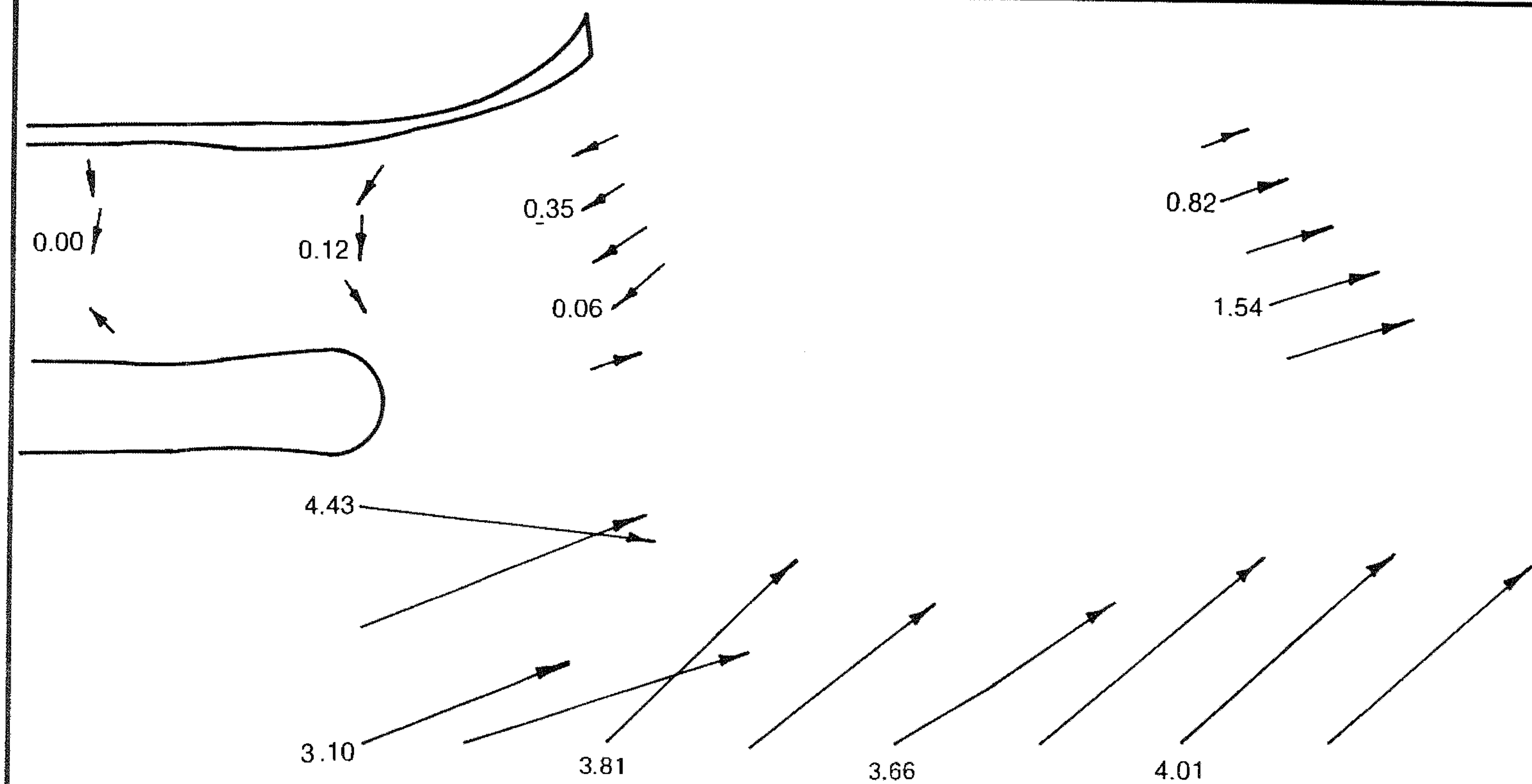
VELOCITY PATTERNS

E = 50

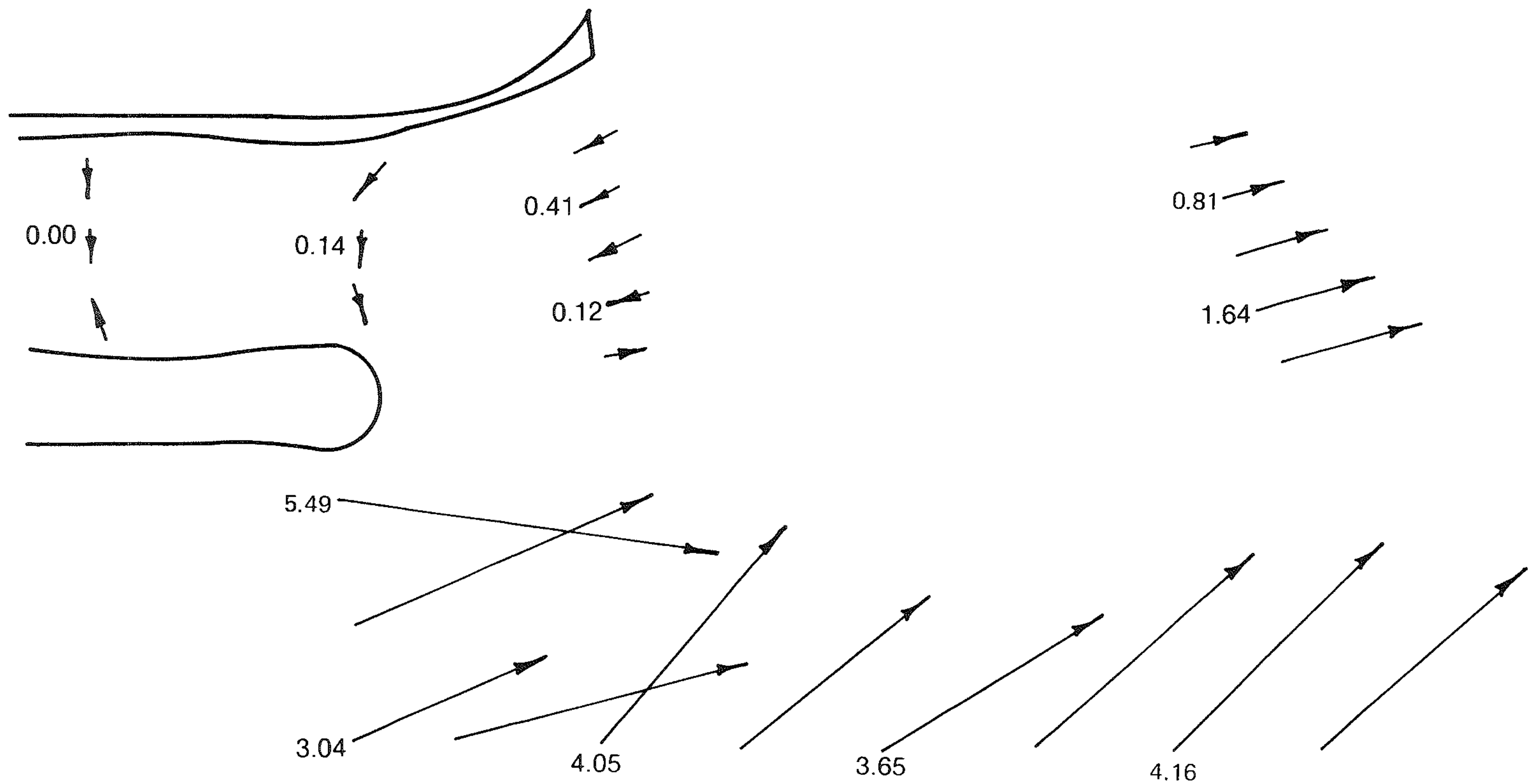


DOWNSTREAM LOCK APPROACH CHANNEL
VELOCITY MAGNITUDES AND DIRECTIONS
Exx = 50 Exy = 50
ORIGINAL DESIGN

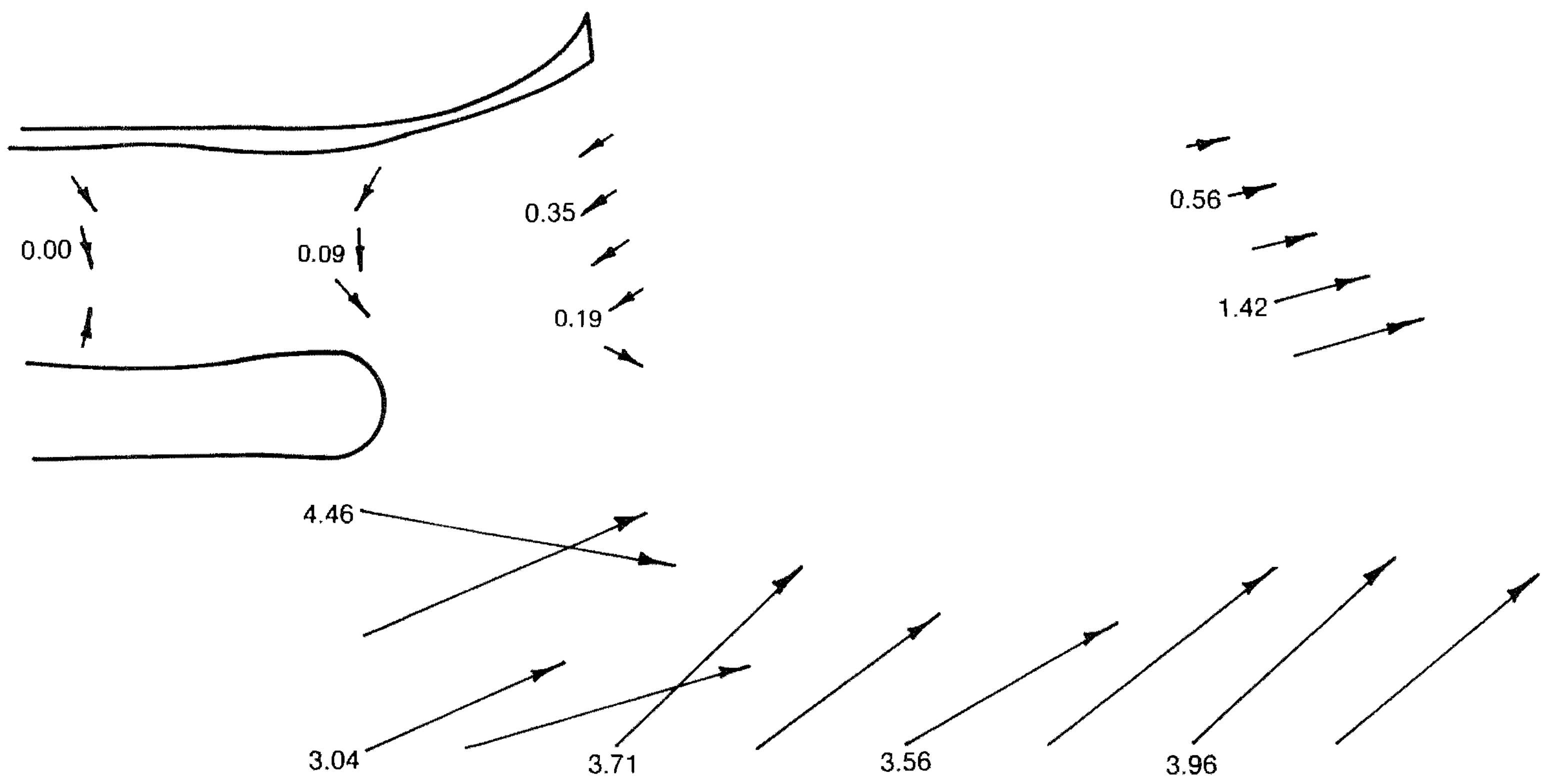
PLATE 8



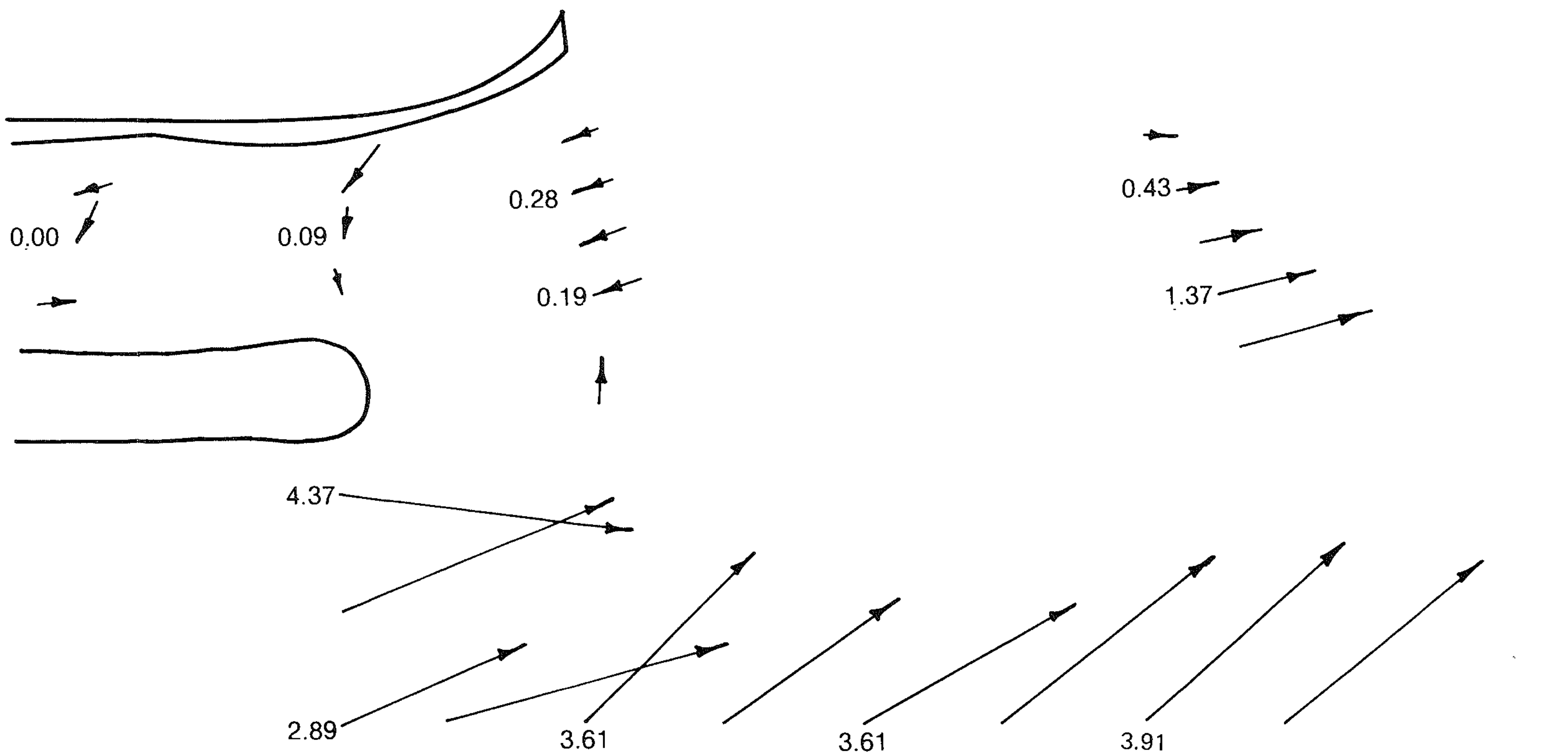
DOWNSTREAM LOCK APPROACH CHANNEL
VELOCITY MAGNITUDES AND DIRECTIONS
Exx = 25 Exy = 25
ORIGINAL DESIGN



DOWNSTREAM LOCK APPROACH CHANNEL
VELOCITY MAGNITUDES AND DIRECTIONS
Exx = 15 Exy = 15
ORIGINAL DESIGN

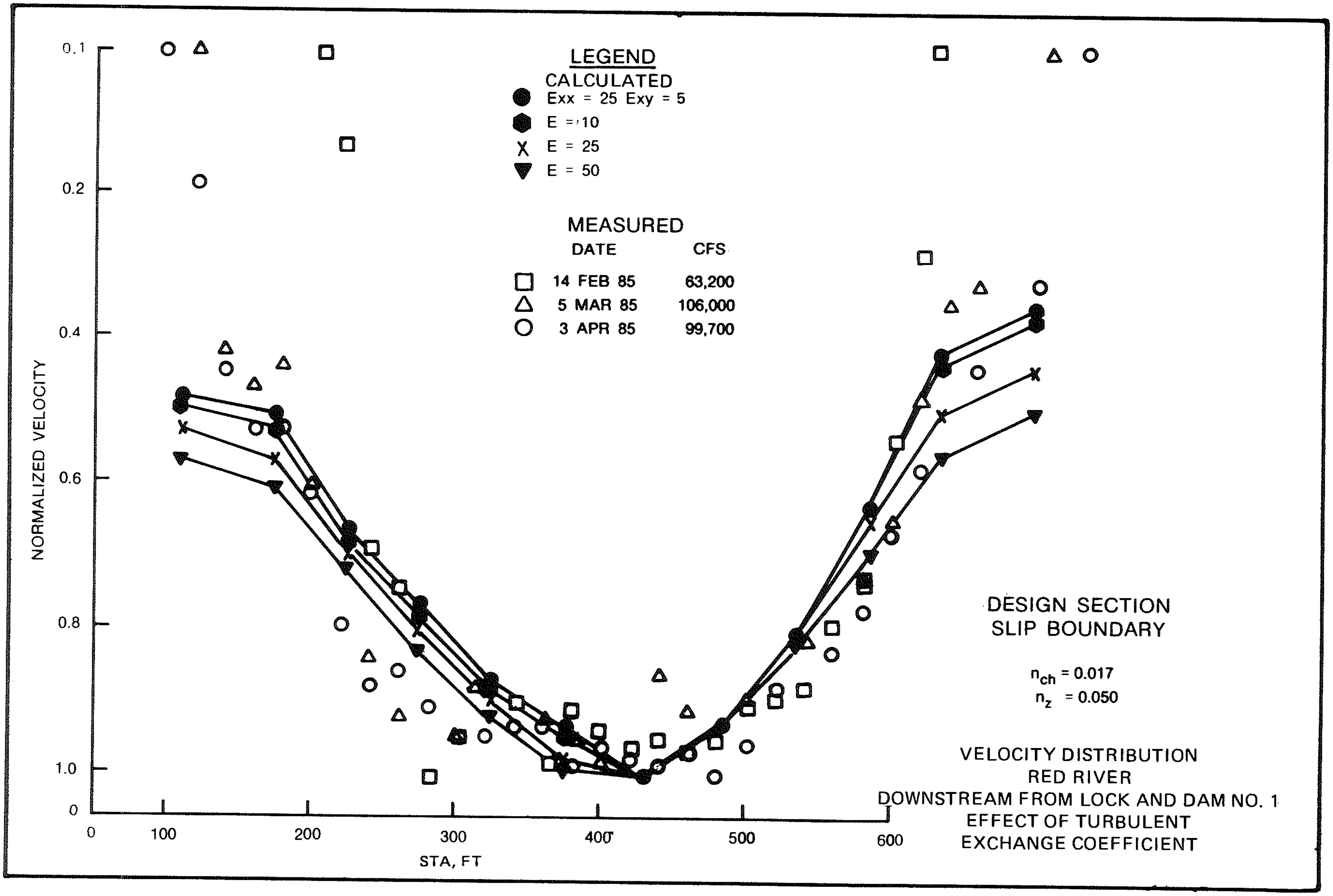


DOWNSTREAM LOCK APPROACH CHANNEL
VELOCITY MAGNITUDES AND DIRECTIONS
Exx = 10 Exy = 10
ORIGINAL DESIGN

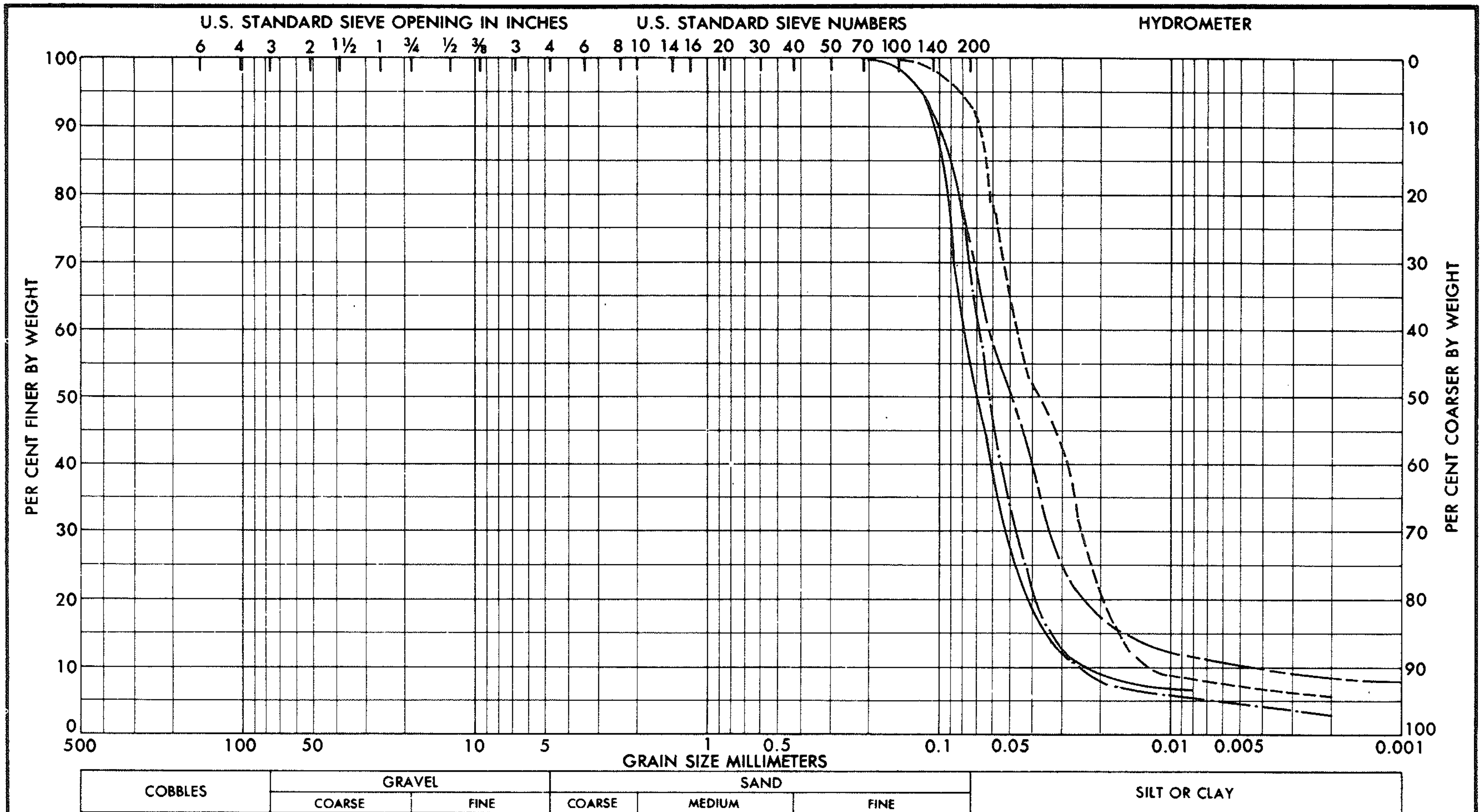


DOWNSTREAM LOCK APPROACH CHANNEL
VELOCITY MAGNITUDES AND DIRECTIONS
Exx = 25 Exy = 5
ORIGINAL DESIGN

PLATE 12



VELOCITY DISTRIBUTION
RED RIVER
DOWNSTREAM FROM LOCK AND DAM NO. 1
EFFECT OF TURBULENT
EXCHANGE COEFFICIENT



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
————	SURFACE	UPSTREAM NOV 84				
—·—	10 FT	DOWNSTREAM AUG 85				
-------	SURFACE	DOWNSTREAM AUG 85				
— — —	SURFACE	DOWNSTREAM LOCK GATE MAR 85				

PROJECT	LOCK AND DAM NO. 1
	RED RIVER WATERWAY
AREA	
BORING NO.	
DATE	

GRADATION CURVES

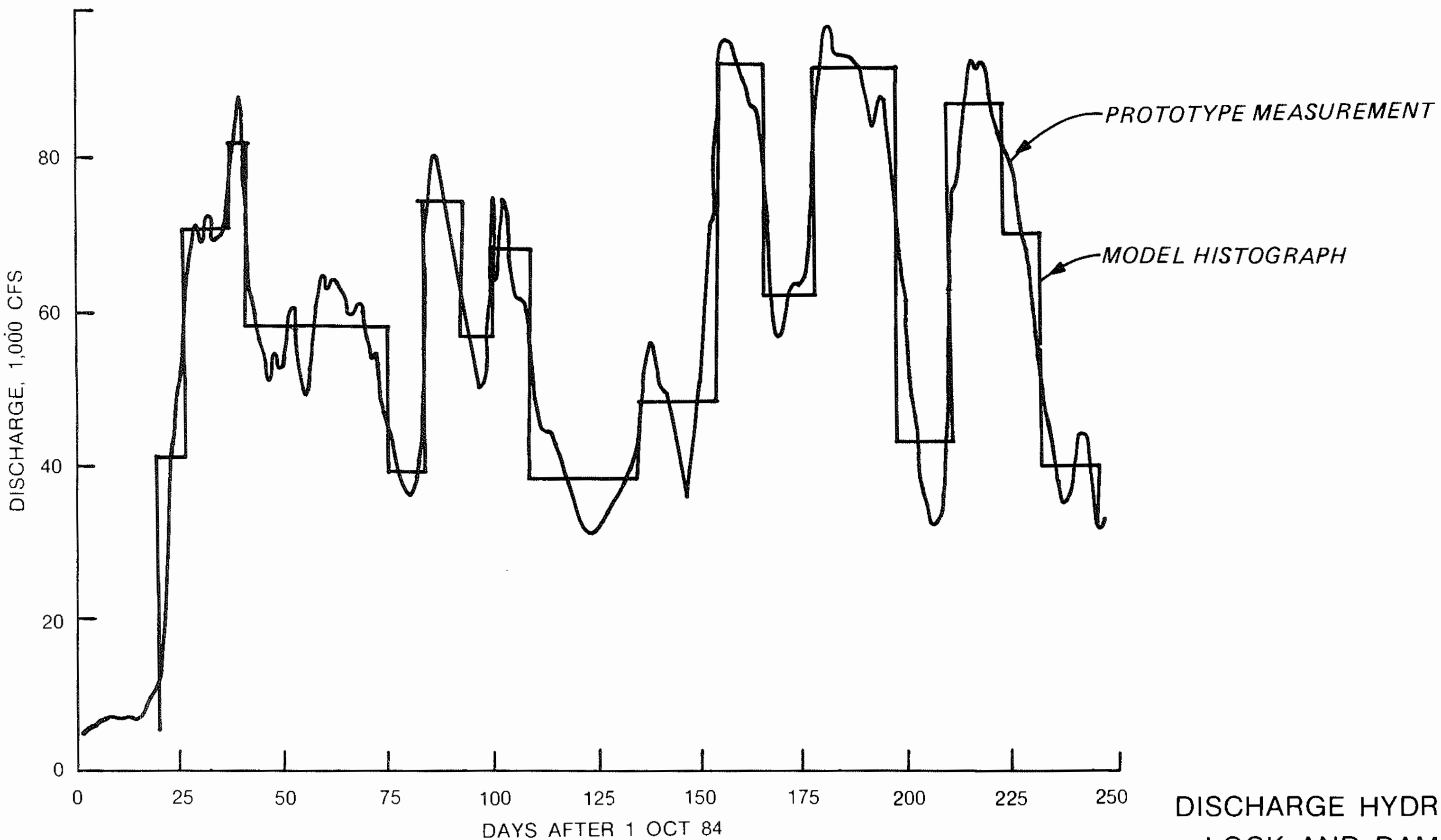
PLATE 13

ENG FORM 2087
1 MAY 63

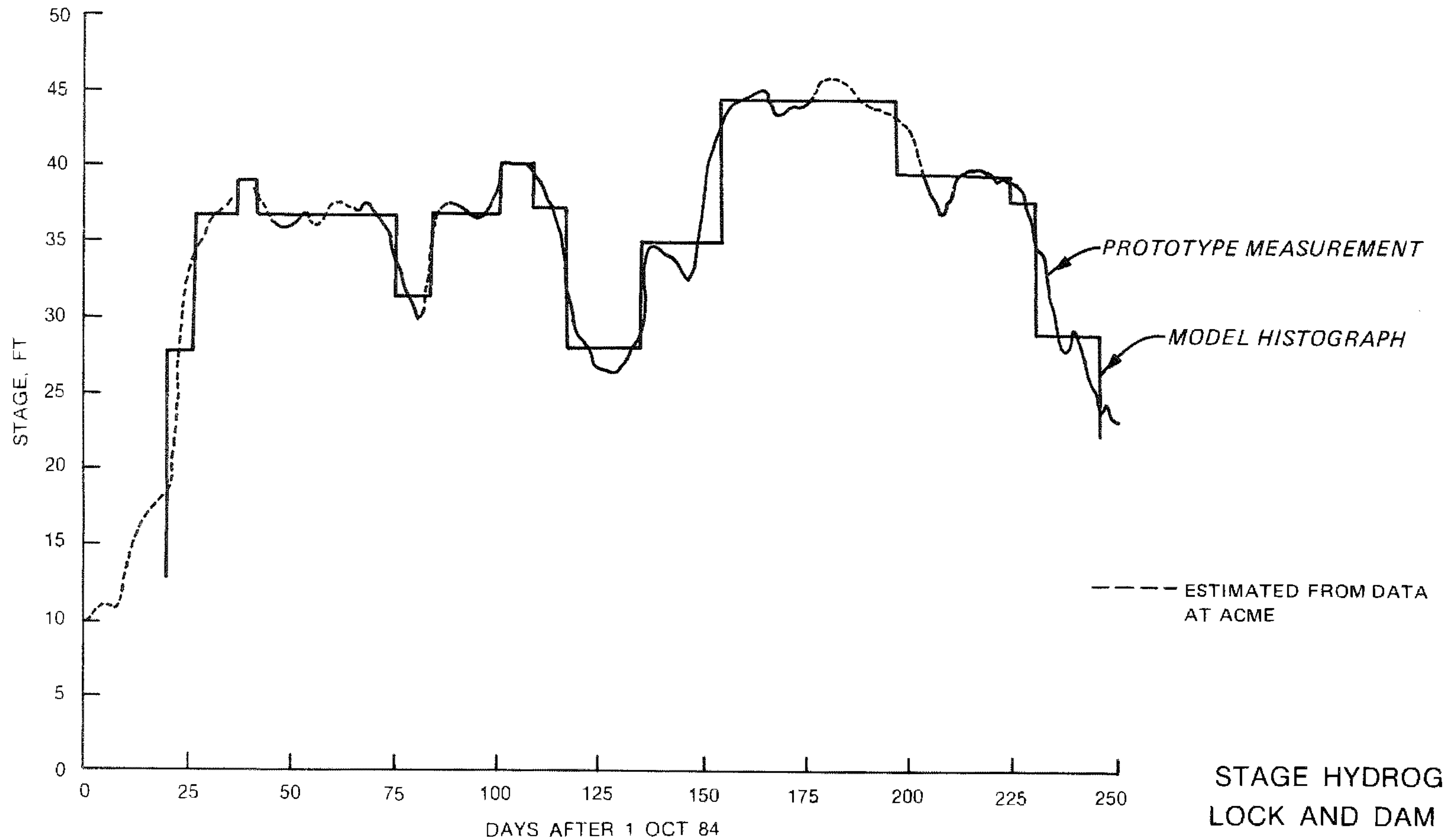
REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.

U.S. GOVERNMENT PRINTING OFFICE . 1963 OF—709-126

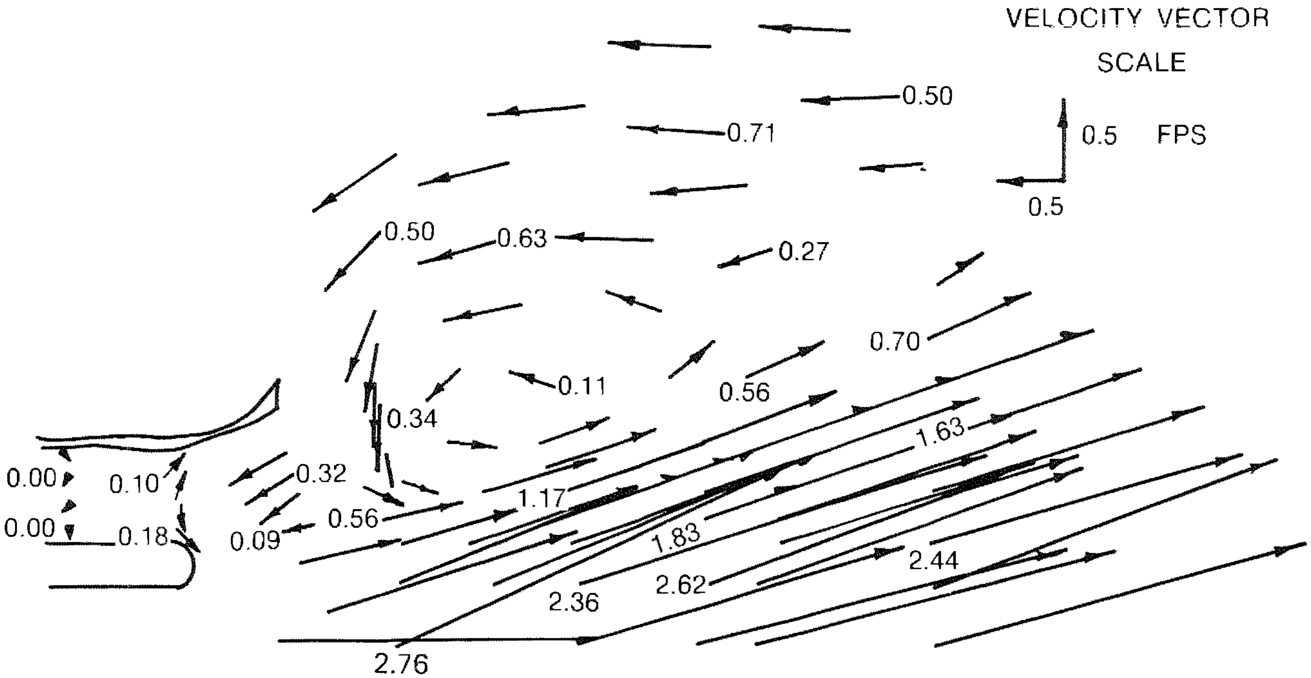
PLATE 14



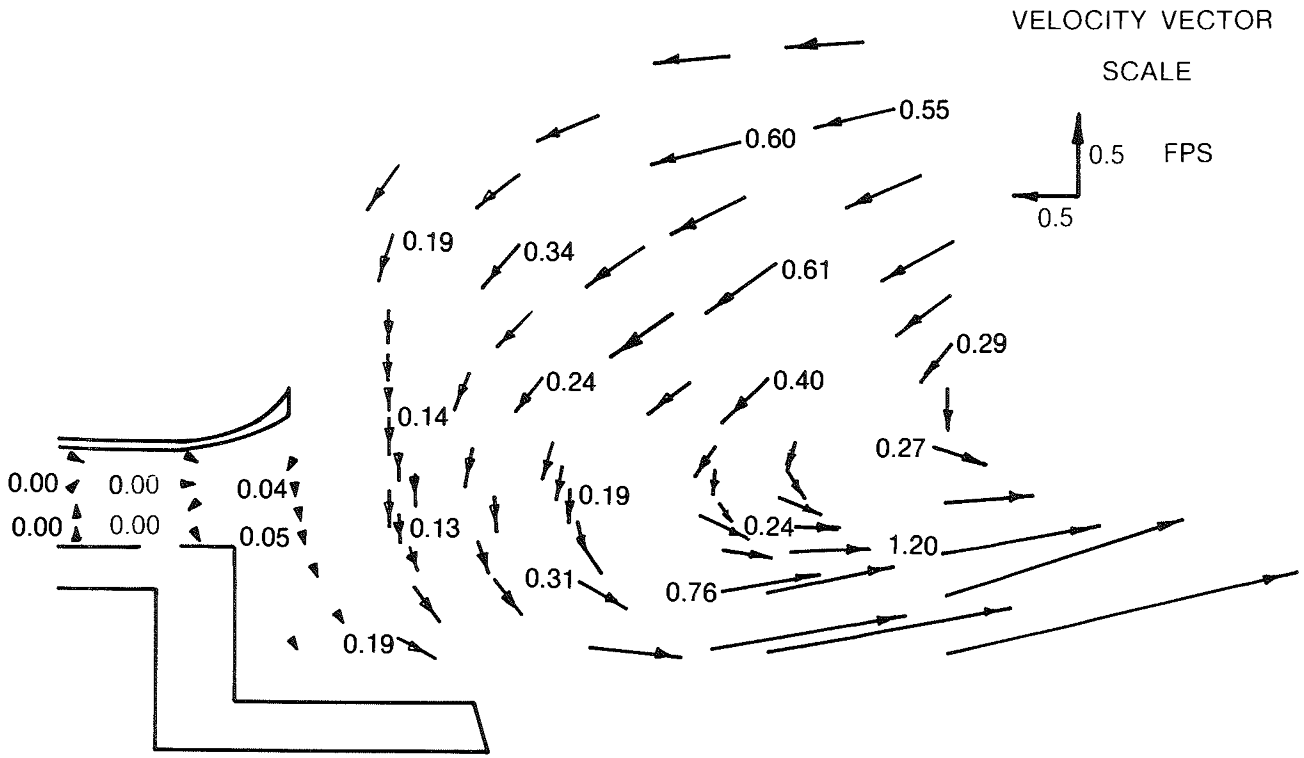
DISCHARGE HYDROGRAPH
LOCK AND DAM NO. 1
1 OCTOBER 1984 -
4 JUNE 1985



STAGE HYDROGRAPH
LOCK AND DAM NO. 1
1 OCTOBER 1984 -
4 JUNE 1985

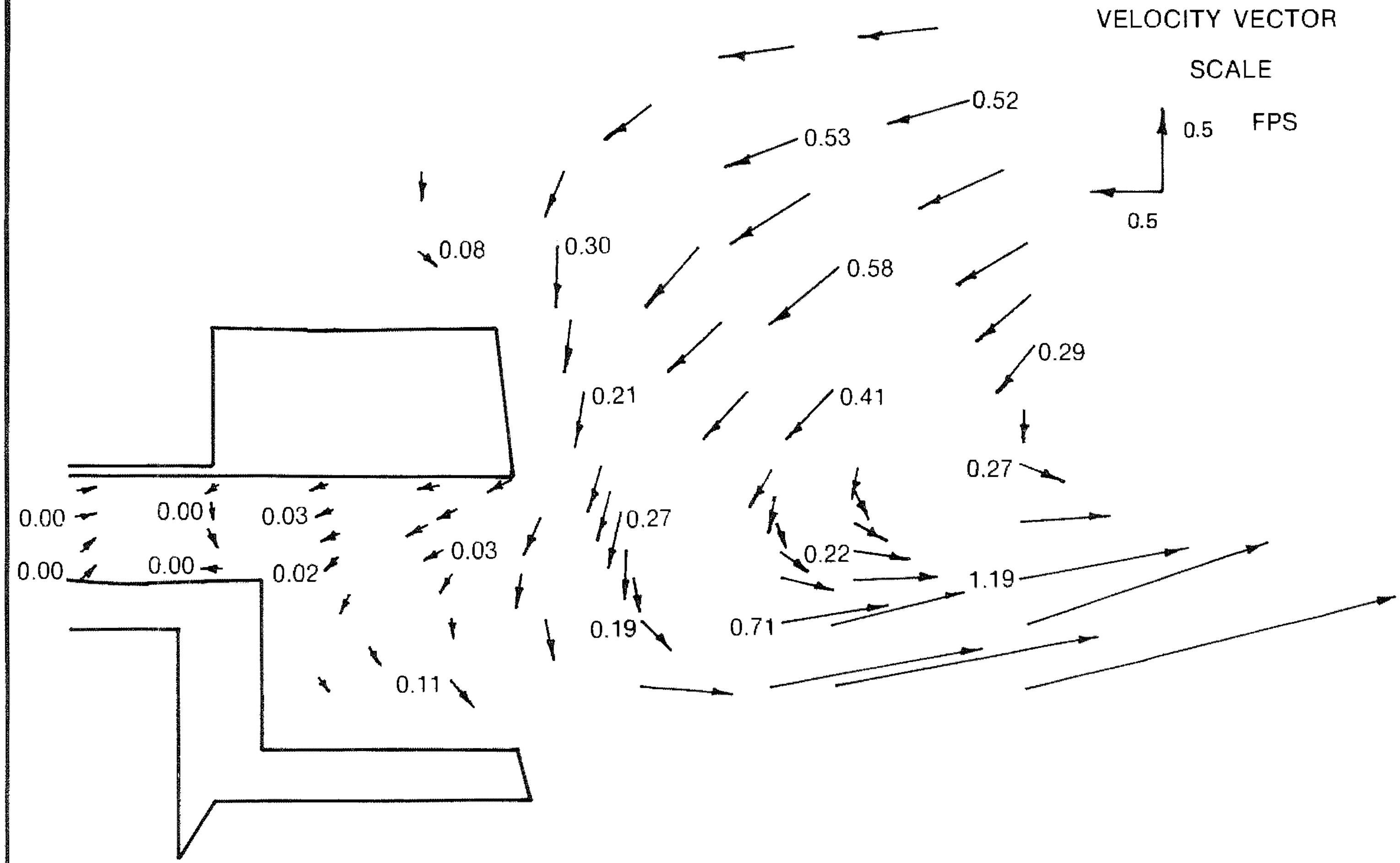


VELOCITY PATTERNS
DOWNSTREAM LOCK APPROACH CHANNEL
EXISTING CONDITIONS, E = 25

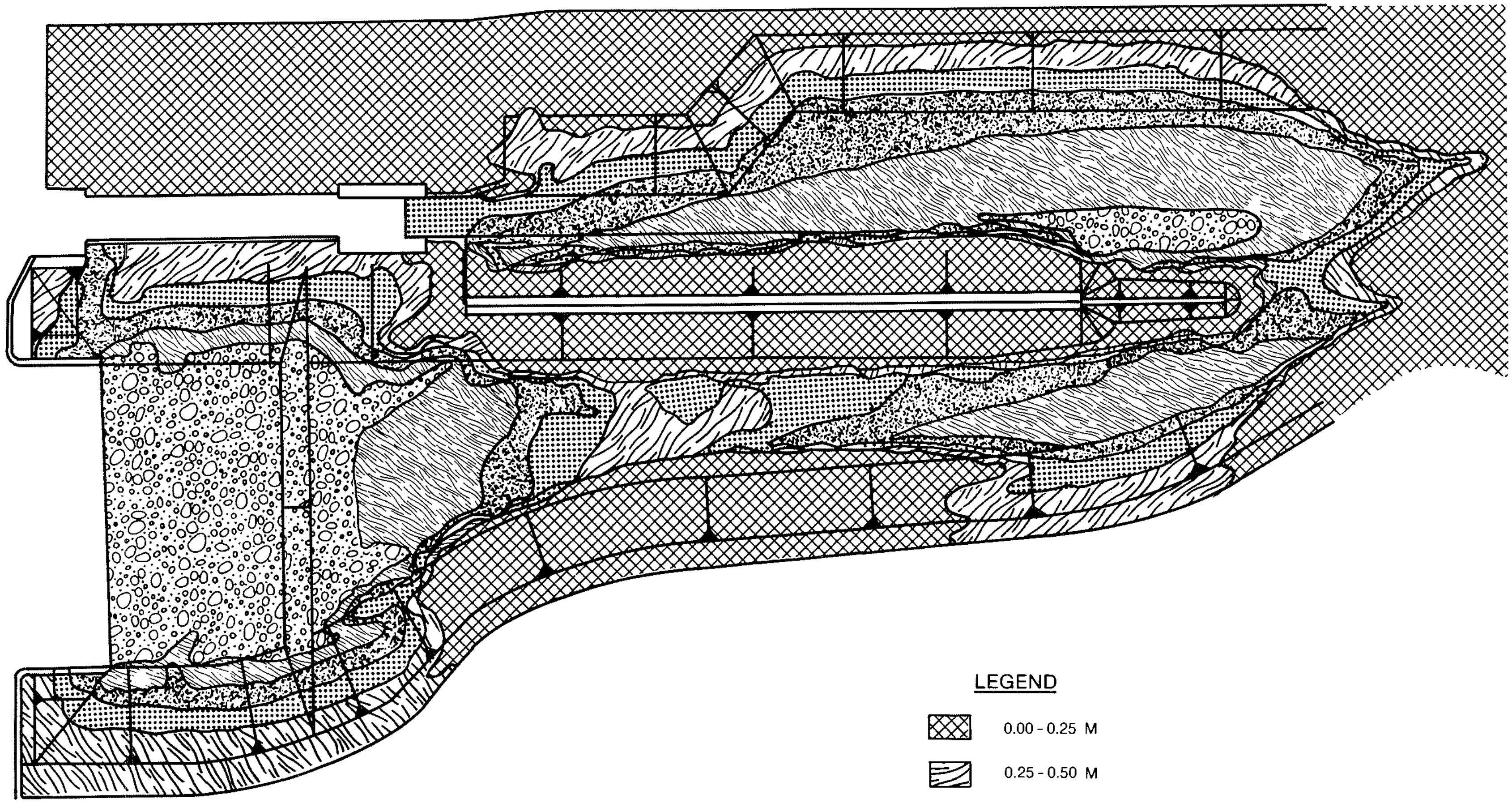


VELOCITY PATTERNS
DOWNSTREAM LOCK APPROACH CHANNEL
ALTERNATIVE NO. 1, E = 25

PLATE 18



VELOCITY PATTERNS
DOWNSTREAM LOCK APPROACH CHANNEL
ALTERNATE NO. 2, E = 25



LEGEND

0.00 - 0.25 M

0.25 - 0.50 M

0.50 - 0.75 M

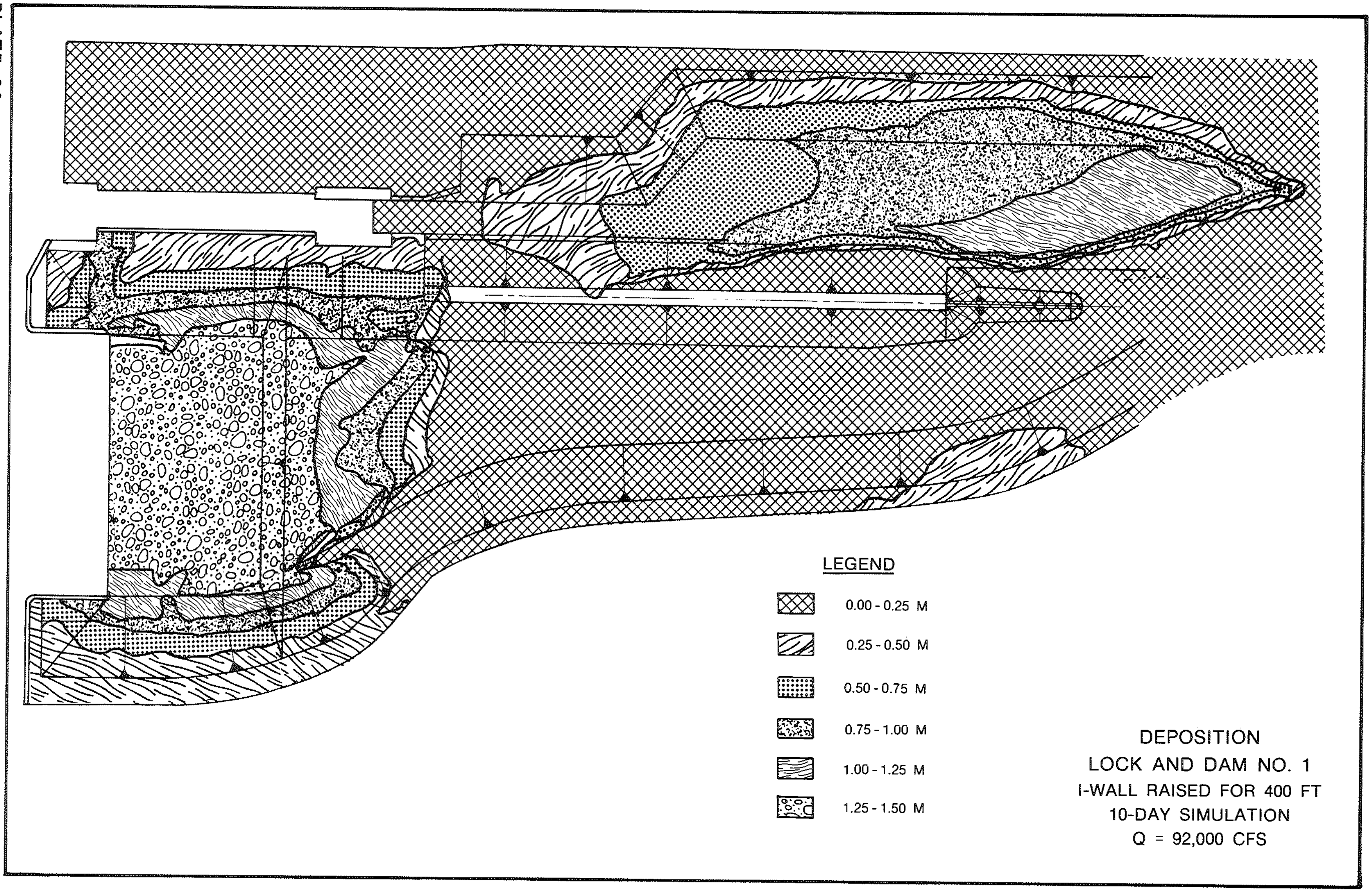
0.75 - 1.00 M

1.00 - 1.25 M

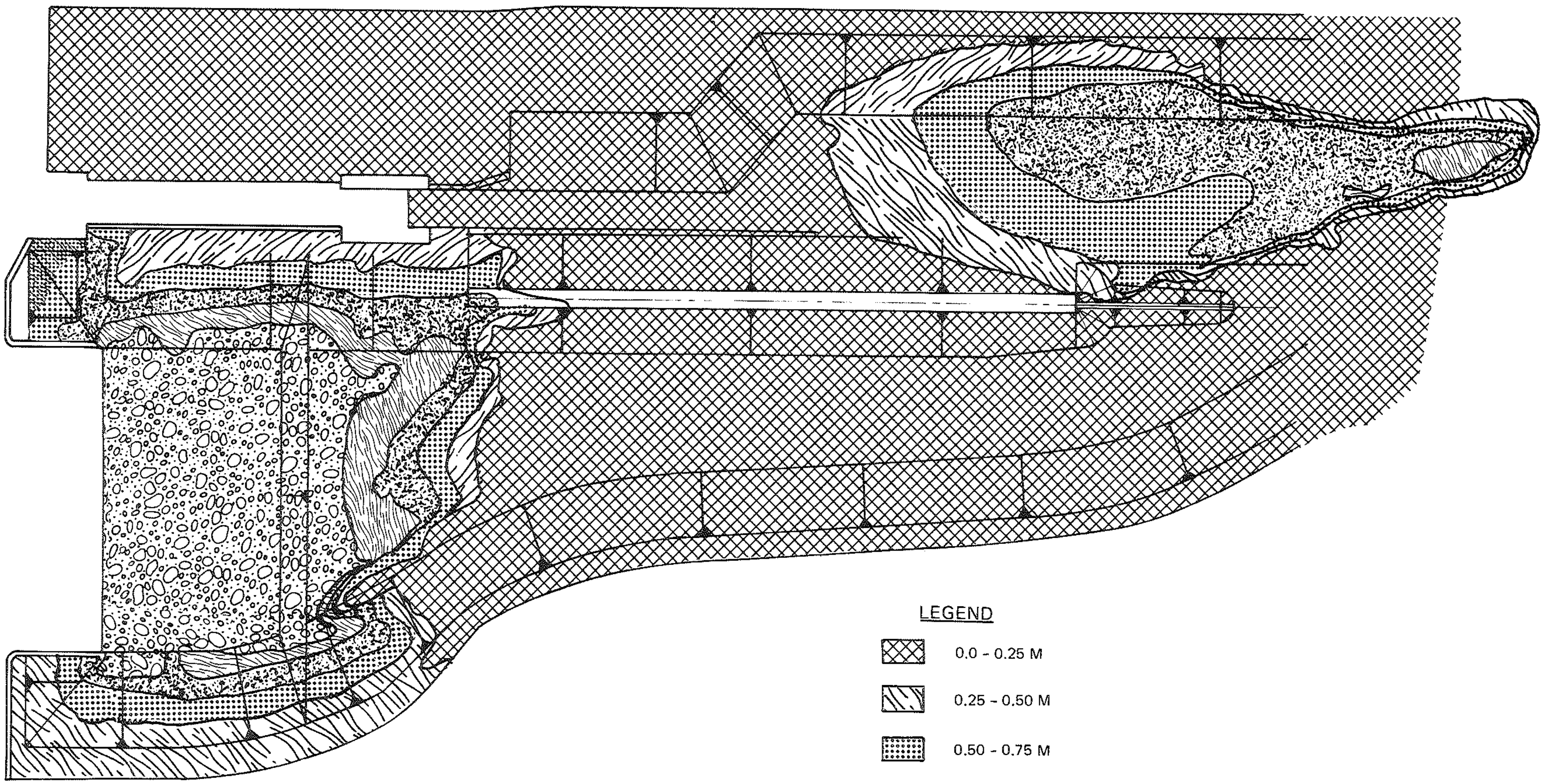
1.25 - 1.50 M

DEPOSITION
LOCK AND DAM NO. 1
EXISTING CONDITIONS
10-DAY SIMULATION
Q = 92,000 CFS

PLATE 20



DEPOSITION
LOCK AND DAM NO. 1
I-WALL RAISED FOR 400 FT
10-DAY SIMULATION
Q = 92,000 CFS



DEPOSITION
LOCK AND DAM NO. 1
I-WALL RAISED FOR 1,300 FT
10-DAY SIMULATION
Q = 92,000 CFS

PLATE 21