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INTERACTION OF RIVER HYDRAULICS AND MORPHOLOGY WITH RIVERINE DR--ETC(U)

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DISSERTATION

INTERACTION OF RIVER HYDRAULICS AND MORPHOLOGY WITH RIVERINE DREDGING OPERATIONS

Submitted by

Peter F. Lagasse

In partial fulfillment of the requirements

for the Degree of Doctor of Philosophy

Colorado State University

Fort Collins, Colorado

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WE HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER OUR SUPERVISION

BY Peter F. Lagasse

ENTITLED INTERACTION OF RIVER HYDRAULICS AND MORPHOLOGY WITH RIVERINE DREDGING OPERATIONS

BE ACCEPTED AS FULFILLING IN PART REQUIREMENTS FOR THE DEGREE OF

DOCTOR OF PHILOSOPHY

Committee on Graduate Work

D. B. Simons
Adviser

Head of Department

ABSTRACT OF DISSERTATION

INTERACTION OF RIVER HYDRAULICS AND MORPHOLOGY
WITH RIVERINE DREDGING OPERATIONS

The current widespread concern over the wise utilization of the nation's natural resources includes questions relative to the nature and significance of the environmental impact of riverine dredging operations. It must be assumed, however, that dredging of large volumes of material from the nation's rivers will continue. Solution to the conflicts between continued dredging requirements and growing environmental constraints requires in part, a fundamental understanding of the geomorphic and hydraulic response of a river system to the dredging process. In addition, the application of basic principles of river mechanics must be considered in developing more acceptable dredged material disposal techniques.

The objectives of this study are to determine the interaction of riverine dredging operations with the morphology and hydraulics of a river system; to examine current open water disposal practices in relation to river morphology; and to investigate the feasibility of disposing dredged material in the main channel region of the river.

In the primary study area, the Mississippi River above Cairo, Illinois, construction of contraction works and navigation dams has taken place concurrently with dredging, and each has simultaneously affected water and sediment transport characteristics of the river. Consequently, the analysis of this study establishes, first, the morphology of the natural river, then, the combined effects of development activities on the river, and, finally, the response of the system to the activity of primary concern, dredging and disposal operations.

The interaction of dredging operations with the hydraulics and morphology of the river system is determined by an analysis based on the hydraulics of dredged cuts in alluvium, results of both prototype experimental dredging and hydraulic model investigations, and the geomorphic principles of river mechanics. The stability of a dredged cut and the consequences of operational policies such as overdredging are examined in detail. The dredge is also viewed as an agent for morphologic change, providing the river engineer with a means of accelerating morphologic processes in support of river development programs.

Dredging requirements are correlated with geomorphic parameters in the study area. These are then used as indicators of dredging problem areas and are applied to a typical field problem, demonstrating that a basic geomorphic analysis provides the river engineer with a methodology applicable to the solution of a variety of problems related to navigation channel maintenance.

A comparison of the environmental impacts of dredged material disposal with the geomorphic and hydraulic implications of open water disposal reveals areas of serious conflict. A one-dimensional mathematical model is used to investigate the feasibility of disposing dredged material in the main channel or thalweg region of the river as an alternative to the current practice of bankline disposal. Results indicate that the concept of thalweg disposal offers a viable alternative to both long-term and emergency disposal requirements.

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Symbol	Definition
A	Cross-sectional flow area
A_d	Volume of deposition of sediment on unit length of channel bed
A_f	Surface area of floodplain
A_x^y	Term which represents departure from a prismatic channel $(\partial A/\partial x)_y$
a	A distance above the bed (usually the bed layer)
C	Concentration of suspended sediment or Chezy resistance coefficient
C_a	Sediment concentration at point "a" above the bed
C_b	Sediment concentration at or near the riverbank
C_F	Fine-material concentration
C_s	A sediment parameter or mean sediment concentration on a volume basis
C_T	Concentration of bed-material discharge
c	A constant or time averaged concentration at a point
D	Depth of flow
D_ℓ	Dynamic contribution of lateral discharge, $(q_\ell V_\ell/Ag)$
d	Representative fall diameter of sediment particle
d_a	Diameter of sediment particles for which "a" percent are finer
d_{50}	Median diameter of sediment particles
F	Width to depth ratio (W/D)
f	Darcy-Weisbach friction factor
f_s	Seepage force in the bed of the stream
g	Acceleration of gravity
h	Water depth

Symbol	Definition
h_f	Head loss due to friction
I	Einstein's integration of the suspended-load relationship, I_1 and I_2
i_b	Fraction of contact load for given grain size
i_s	Fraction of suspended load for given grain size
i_t	Fraction of total load for given grain size
k	Colby's correction factors, k_1 k_2, and k_3
k_s	Height of roughness element in the bed (d_{65} for sand bed)
M	Percent silt-clay in bed or banks
m	Mass
n	Manning's roughness coefficient
n_b	A roughness coefficient
$n\Delta$	Width of a dredged cut in terms of its depth, Δ, where n is an integer multiplier
P	Wetted perimeter
P_E	Einstein's transport parameter
p	Pressure or the volume of sediment in a unit volume of bed sediment
Q	Water discharge
Q_b'	Water discharge quantity determining bed-load transport
Q_s	Total sediment discharge
q	Water discharge per unit width
q_b	Contact-load transport per unit width
q_ℓ	Lateral inflow per unit length of channel
q_n	Colby's uncorrected sediment discharge per unit width
q_s	Sediment transport per unit width or lateral sediment inflow per unit length of channel
q_t	Total-load transport per unit width

Symbol	Definition
R	Hydraulic radius (A/P)
RM	River Miles along the Upper Mississippi River measured as distance above Cairo, Illinois
r	Radius
S	Slope of channel bed or slope of energy grade line
S_b	Percent silt-clay in banks
S_c	Shape factor of the stream cross section or percent silt-clay in the bed
S_f	Friction slope
S_o	Bed slope at a reference station
S_p	Shape factor of sediment particles
S_R	Shape factor of a river reach
S_s	Specific gravity of sand
S_x	Bed slope at a station
s	Sinuosity
t	Time
V	Mean flow velocity (Q/A)
V_ℓ	Velocity component of lateral inflow in the main flow direction
V_Θ	Radial velocity
v	Velocity at a point
v_*	Shear velocity ($\sqrt{gRS}$)
W	Width of stream
x	Horizontal distance along a channel or Einstein's multiplication factor
y	Water depth
z	Elevation of water surface or bed, or the Rouse Number ($\omega/\beta\ \kappa\ v_*$)

Symbol	Definition
α	Abrasion coefficient
β	A wear or sorting coefficient, a coefficient relating diffusion coefficients, or momentum correction coefficient
γ	Specific weight of fluid
Δ	Depth of a dredged cut
κ	von Karman velocity coefficient
μ	Apparent viscosity of water-sediment mixture or dynamic viscosity of fluid
ρ	Density of water-sediment mixture or mass density of fluid
ρ_b	Density of sediment forming the bed
ρ_s	Density of sediment
ρ_w	Density of water
σ	Gradation of bed material
τ_c	Critical shear at which motion initiated
τ_o	Shear stress at a boundary
ϕ_*	Intensity of transport for given grain size (Einstein)
ψ_*	Intensity of shear for given grain size (Einstein)
ω	Particle fall velocity

Chapter 1

INTRODUCTION

In the Transactions of the American Society of Civil Engineers, December 1898, J.A. Ockerson noted in regard to navigation on the Mississippi River: "the necessity for some suitable device for removal of sand bars has long been felt. Some thirty years ago a board of engineers recommended that a prize of $100,000 be offered for the best device for removing obstructing sand bars from navigable streams." Although Congress did not carry out this recommendation by making the necessary appropriation, a number of inventions were tendered to the Government for use. Among these was a jet device to "make each steamboat independent of any general improvement of the channel by providing suitable jets which will enable the boat to work its own way through the bars" (Ockerson, 1898).

In urging the utility of providing individual river craft with water jets, the inventor provided the following rationale: "It cannot be expected that the Government will every year spend thousands of dollars to remove sandbars which re-form at every flood. Those interested in river navigation ought to make themselves independent of such obstructions and of Government aid, but they will not entertain the idea until forced to" (Ockerson, 1898). This individual's singular lack of success in relieving the Government of responsibility for maintaining the nation's navigable waterways is reflected in the current scope of Corps of Engineers' programs for "removal of sandbars" and developing and maintaining navigation channels.

In fulfilling its mission relative to such programs the Corps of Engineers is responsible for the dredging of large volumes of sediments from the nation's harbors and waterways each year. Annual quantities average 300,000,000 cubic yards (cy) in maintenance dredging and 50,000,000 cy in development (new work) dredging, with total annual costs in excess of $150,000,000. Over 19,000 miles of waterways and 1,000 harbor projects are currently being maintained by the Corps in support of the nation's waterborne commerce. As indicated by Ockerson, the Mississippi River has long been an area of concern in regard to maintenance of navigable depths. Currently about 57,000,000 cy or 19 percent of the Corps' average annual maintenance dredging requirements, consists of dredging in the Mississippi River Districts of St. Paul, Rock Island, St. Louis, Memphis, and Vicksburg (Boyd et al., 1972). Jointly, these Districts are responsible for maintaining navigable depths in the Mississippi River from St. Paul, Minnesota to Baton Rouge, Louisiana.

The current widespread concern over the wise utilization of the nation's natural resources must include questions relative to a program of this magnitude, particularly in regard to the nature and significance of the environmental impact of dredging and disposal operations. Despite growing concern over the environmental impacts of dredging, it must be assumed that dredging of large volumes of material will continue, primarily because of a current lack of acceptable alternatives in creating and maintaining navigable waterways. The importance of waterborne commerce to the nation's economy supports this contention. There has been an 85 percent increase in total tonnage of waterborne commerce in the 20 year period 1950-1970, reaching a total in excess of 1.5 billion tons in 1970. Projections indicate a continuing important role for waterborne commerce in the nation's economic development (Boyd et al., 1972; Mohr, 1974).

Concurrently, it must also be anticipated that increasing environmental concern will result in significant controls on dredging methods and dredged material disposal procedures. The Environmental Policy Act of 1969 requires a detailed statement of the environmental impact of proposed new navigation projects and projects requiring maintenance dredging. In addition both the 1972 Amendments to the Water Quality Control Act and the Ocean Dumping Bill require development of criteria related to dredging activities (O'Neal, 1972). At present, many dredges in the United States sit idle as projects are stalled by Environmental Protection Agency objections or by the lengthy preliminary studies now required (Turner and Fairweather, 1974).

Solution to the dilemma of expanding requirements and growing constraints will require a fundamental understanding of the geomorphic and hydraulic response of a river system to the actual dredging process. In addition, the application of basic principles of river mechanics must be considered in developing more acceptable dredged material disposal techniques and in selecting disposal sites which minimize environmental impacts.

In the past, dredged material disposal sites have been selected by the criteria of physical capabilities of the dredge plant, minimal disposal costs, and minimizing environmental impact. The first two constraints of capability and cost have often been at variance with environmental considerations. In general, decisions concerning disposal techniques have been based on economic considerations. Significant departure from this criteria in the future will require increased capital investment to upgrade dredging plant capabilities, acceptance of increased disposal costs, or development of innovative dredging and disposal techniques.

1.1 Objectives

The purpose of this study is to analyze the response of a river to the dredging process and to apply principles of river mechanics to the problem of minimizing environmental impacts of dredged material disposal. To this end, the following specific tasks are undertaken:

1. Determine the interaction of riverine dredging operations with the morphology and hydraulics of a river system.
2. Examine the current practice of bankline and island disposal of dredged material in relation to river morphology and hydraulics, and investigate the feasibility of disposing dredged material in the main channel region of the river.

1.2 Concepts and Constraints

Although Webster gives the primary definition of a dredge as "an oblong iron frame, with a bag net attached, dragged over the sea bottom, used especially for gathering shell fish," an engineer generally associates a different device and a different purpose with the term. To the engineer, a dredge is a "floating excavating machine, and the process of removing subaqueous material is termed dredging" (Simon, 1920). With emphasis on the objectives of dredging, a better definition would be "an earth-moving machine specialized to remove bottom material from under water to increase the water depth or gain the bottom material" (Mohr, 1974). More precisely, dredging is a process by which sediments are removed from the bottom of streams, lakes, and coastal waters, transported by ship, barge, or pipeline, and discharged in open water or on land (Boyd et al., 1972). Dredging is performed to develop, extend or maintain waterways, or to obtain material for fill or commercial use.

It is useful to classify dredging operations by disposal location. Open water disposal involves the disposition of dredged materials into open ocean, bays, estuaries, and inland rivers and lakes. This definition can be expanded to include those materials placed on beaches, marshes, and along banklines where they are subject to the influence of tides, river stage fluctuations, or are readily washed back into the water by rainfall. Conversely, land or terrestrial disposal implies placing the dredged material beyond the influence of even the highest tides or river stages, and in confined or unconfined locations where rainfall runoff cannot transport significant quantities back to the water (Boyd et al., 1972).

An indication of the quantities of dredged material involved relative to disposal locations and environmental regions can be obtained from the national overview of the Corps' average annual maintenance dredging requirements presented in Table 1. Two-thirds of the material produced by maintenance dredging programs is disposed of in open water, with the remaining one-third disposed of on land. About 45 percent of the material disposed of in open water enters the estuarine or marine environments and three percent enters a lacustrine environment, predominantly in the Great Lakes Region. The remaining 52 percent enters the riverine environment.

In very general terms, materials dredged and disposed of in inland waterways are predominantly sand and gravel. For years the nation's navigable rivers have provided a supply of these materials for construction purposes. In lakes, harbors and areas of the coastal zone where the transport capacity of the water is low, dredged materials are more often silts and clays. Because of industrial development and population concentrations contiguous to navigable waterways, the materials dredged from some harbors, estuaries, and channels are polluted. Of the total amount of material disposed of annually in open water, 30 to 35 percent can be classified as polluted using current EPA criteria. Again, in general terms, inland waterway dredging operations such as those of the Mississippi River Districts produce cleaner dredged material than operations in the estuarine or marine environments where polluted bottom materials are more prevalent (Boyd et al., 1972). This generalization must, of course, be tempered by consideration of the proximity of a given location to industrial or urban centers.

In terms of equipment utilized, hydraulic pipeline dredging accounts for approximately 70 percent of the Corps' average annual maintenance dredging. Dustpan and cutterhead dredges both fall in this category. Since a description of dredge capabilities and operational concepts is presented in a subsequent chapter, it is the purpose here to highlight only the relative importance of various dredge types. Dustpan dredges account for about 40,000,000 cy of dredging annually and are used almost exclusively for channel maintenance dredging in the Mississippi River. Cutterhead dredges are used throughout the United States and contribute approximately 166,000,000 cy of maintenance dredging annually. A third dredge type, the hopper dredge, is used primarily for deeper water operations in the Great Lakes Region and in the coastal zone, providing annually about 70,000,000 cy of maintenance dredging. The balance of the annual maintenance dredging requirement is accomplished by a variety of quantitatively less important dredge types, including dipper, clamshell, bucket, and sidecaster dredges (Boyd et al., 1972).

Much of the concern over the actual dredging process is related to the direct destruction of benthic (bottom-dwelling) organisms. Similarly, most of the concern over the disposal of dredged material is directed toward the effects of open water disposal of polluted bottom sediments on water quality and aquatic organisms. Because of this concern over the open water disposal of polluted sediment, a trend toward land disposal has developed, but here too a myriad of environmental problems arise. Land disposal often involves marshlands or wetlands peripheral to river channels and estuaries, and these areas are highly productive biologically.

Problems relative to these aspects of the dredging and disposal process are currently the subject of an intensive five year, $30,000,000 research effort by the Environmental Effects Laboratory of the Corps of Engineers Waterways Experiment Station. The Dredged Material Research Program, authorized by the River and Harbor Act of 1970, has the objective "to provide, through research, definitive information on the environmental impact of dredging and dredged material disposal operations, and to develop, technically satisfactory, environmentally compatible, and economically feasible dredging and disposal alternatives..." This research program is subdivided into four projects: Aquatic Disposal, Habitat Development, Land Disposal and Equipment, and Productive Uses (Boyd et al., 1972). An analysis of the research areas under each project indicates that this research effort is strongly oriented toward the environmental concerns of: direct biological effect of the dredging process, open water disposal of polluted dredged material, particularly in the estuarine, marine, and lacustrine environments, and the problems of confined and unconfined land disposal.

With the foregoing concepts established the subject area of this study can now be precisely delineated. Specifically, this study addresses the hydraulic and geomorphic response of a river to the actual dredging process, and the problem of open water disposal in the riverine environment. The effects of development (new work) dredging, maintenance dredging, and dredging to produce construction materials (sand and gravel mining) are considered. Open water disposal includes dredged material placed on islands, marshes, and along river banks at locations where these materials are subject to the influence of river stage fluctuations, or are readily washed back into the river by rainfall. Quantitatively, this includes about 50 percent of the dredged material disposed of annually in open water under the Corps' maintenance dredging program (Table 1). As cutterhead and dustpan hydraulic pipeline dredges accomplish most riverine dredging, these are the only dredge types considered in detail.

Problem areas currently under study by the Waterways Experiment Station's Environmental Effects Laboratory are not considered directly. Consequently, the direct biological effects of the dredging process, the impact of disposal of polluted materials, and the environmental problems of land disposal are not within the purview of this study. Hopper dredge operations associated with estuary and coastal channel maintenance are not considered in detail, although operational concepts developed for estuarine dredging have some application to riverine dredging problems. In relation to open water disposal of unpolluted dredged material, the environmental impacts of primary concern are the processes of filling sloughs, chute channels and backwater areas, and the concomitant loss of diversified habitat in these biologically productive regions.

1.3 Selection of a Study Area

The response of a river to man's development is extremely complex. It is probable that each of the works of man influences to some degree all of the morphologic and hydraulic parameters of a river system. This is true relative to dike construction, bankline revetment, construction of cutoffs, installation of locks and dams for navigation, and may also be true of dredging.

To analyze the response of a river to dredging and disposal operations, it is necessary to establish first, the morphology and hydraulics of a selected study area prior to man's intervention. With this as a baseline the response of the river to man's activities can be developed. Finally, an attempt can be made to isolate the response of the system to a selected activity such as dredging and disposal. Unfortunately, man's development of a river seldom progresses in isolated, sequential steps. Construction of contraction works, revetment, and dredging, for example, usually take place concurrently and each simultaneously affects, to some degree, the water and sediment transport characteristics of the system. Consequently, the task of isolating the response of the river to the dredging process is difficult, and conclusions relative to this response must be based to some extent on inference, deduction, and an analysis of secondary effects.

This analysis requires, then, selection of a study area where both detailed, long term hydraulic and geomorphic data, as well as extensive dredging records are available. The Mississippi River from

St. Paul, Minnesota to Cairo, Illinois satisfies each of these requirements. This portion of the Mississippi River falls under the joint responsibility of the St. Paul, Rock Island, and St. Louis District offices of the Corps of Engineers. The reach from St. Paul, Minnesota to St. Louis, Missouri is generally referred to as the Upper Mississippi, and that from St. Louis to Cairo, Illinois, the Middle Mississippi (Figure 1). The hydrographic data available through the District offices is detailed enough to permit time sequencing changes in river morphology through the years 1891, 1930, 1940, and 1971. In addition aerial photography of the river from St. Louis to St. Paul is available for the years 1927 and 1973. Both the Corps of Engineers and the United States Geologic Survey (USGS) maintain numerous long term stage and discharge gaging stations on the Upper and Middle Mississippi. For example, the St. Louis gage records provide stage and discharge data since 1843.

In terms of dredging impact this portion of the Mississippi River provides an excellent study area. Ockerson (1898) and Maltby (1905) both provide summaries of navigation and dredging problems that date back to the 1850's, indicating that maintenance of navigable depths in the region has long been a concern. The Corps District offices at St. Paul, Rock Island, and St. Louis have maintained long term dredging records on their sections of the Mississippi River that detail by year, location and quantity, dredge cuts and disposal sites.

In addition to providing the necessary data, selection of the Mississippi River above Cairo as a study area provides the opportunity to contribute to the solution of significant problems relative to dredging and disposal in the riverine environment. Currently the

St. Paul, Rock Island and St. Louis Districts account for 12,000,000 cy of maintenance dredging annually. While not overwhelming in terms of the Corps' total dredging responsibility, disposal of this volume of dredged material does constitute a major problem for the region. The controversy over establishing and maintaining a minimum nine-foot navigation channel, which is sketched subsequently, and current opposition to the possibility of a 12 foot navigation channel, provide ample testimony to the existence of serious problems. The conflict between dredging requirements and environmental interests on the Upper Mississippi River is underscored by the State of Wisconsin's recent attempt to obtain an injunction which would preclude dredging and disposal operations by the Corps in the State's rivers (U.S. District Court, Opinion and Order, July 1973).

The selection of this region as a study area was influenced by an additional consideration. Navigation depths on the Upper Mississippi River have been achieved through contraction, revetment, dredging, and construction of a series of low dams and locks that extends from St. Paul to St. Louis. Below St. Louis, on the Middle Mississippi River, navigation depths are maintained primarily by contraction dikes, bankline revetment, and dredging, without the use of navigation dams. While the Upper Mississippi has been converted into a series of regulated pools at low water, the Middle Mississippi has remained essentially in an "open" configuration. Use of these two reaches provides the opportunity to analyze dredging impacts on reaches whose hydraulics and morphology differ markedly as a result of man's development.

1.4 Supplemental Data

While detailed analysis of dredging and disposal impacts is restricted to the primary study area, the Upper and Middle Mississippi River, riverine dredging operations are conducted at diverse locations throughout the world. Experimental and field data from locations other than the primary study area provide a valuable supplement to this study.

In the United States, dredging has provided a means of rapidly changing the morphology of a river system in support of development work for the Arkansas River lock and dam system, on the Ohio River, and in conjunction with river regulation on the Lower Mississippi River. Experimental dredging on the Lower Mississippi and experience gained from long-term maintenance dredging programs on the Lower Mississippi, Columbia and Apalachicola Rivers also provide material which supplements data from similar programs in the primary study area. Hydraulic model investigations of navigation and shoaling problems on the Arkansas and Ohio Rivers include limited investigations of the stability of dredged cuts which will supplement data from prototype rivers. An analysis of the effects of gravel mining on the Lower Mississippi River provides valuable data relative to the effects of dredging to obtain construction materials.

In regard to supplementary data from sources outside the United States, valuable experience has been gained from long term maintenance dredging programs on rivers in the U.S.S.R., and in China. The Netherlands Engineering Consultants (NEDECO) have investigated the hydraulic aspects of riverine dredging in connection with river studies to improve navigation conditions on the Niger and Benue Rivers in Nigeria as well as the Rio Magdalena in Columbia, and have developed a comprehensive analysis of development and maintenance dredging on rivers for the East Pakistan Inland Water Transportation Authority. Results of hydraulic model investigations of dredging problems by the Delft Hydraulics Laboratory and a study of siltation problems in trenches dredged for the Benelux tunnel and for pipelines under the Wester-Scheldt are also relevant sources of supplementary data for this study.

1.5 The Dredging Problem in Perspective

In his 1972 annual report the Chairman of the ASCE Task Committee on Maintenance of Navigable Waters indicated that one of the major problems in accomplishing task committee objectives in regard to dredging was "the inability to induce qualified professionals to devote study time to the operation and maintenance activity, considered prosaic by engineers..." He observed further that "many opportunities exist in the maintenance field for those who would challenge the traditional operation by introducing modern technology and improved methodology with emphasis on dredging" (Black, 1972).

In outlining the research needs of the dredging industry Herbich (1968) notes that "the dredging industry is, compared with similar-sized industries, near the bottom of the list in time, effort, and money devoted to study, research, and development." Huston (1968) analyzed the needs of the dredging industry and concluded that they are myriad. "For instance, we need information, and better dissemination of what we have, we need training programs. We need to invest more time, effort and money in study, research and development." In addition "there is a dearth of published, practical, and useful dredging information. On the premise that a profession is known by its literature, dredging might well be eliminated. Its literature is almost nil... Less has been written intelligently about the dredge than any other piece of excavating equipment known to man."

Gower (1967) in a history of dredging refers to a period of stagnation in the development of dredges and dredging technology from 500 AD to 1500 AD, and Black (1973) concludes that "the United States has been in such a period during the past 25-30 years..." Current emphasis on dredging technology, spurred by environmental concerns, indicates that this period of stagnation may well be at an end; however, a void has existed in the application of basic engineering principles to dredging operations. It is intended that this study fill some small part of that void.

An analysis of the response of a river system to dredging and disposal operations requires establishing, first, the morphology and hydraulics of the system prior to man's influence, and second, the response to man's activities occurring both prior to and concurrent with dredging. Chapters 2 and 3 develop the necessary baseline for this analysis. A brief sketch of the geologic history of the study area in Chapter 2 is followed by a description of the morphology of the Upper and Middle Mississippi River in the "natural" state in Chapter 3. A discussion of the applicable principles of river mechanics, and an analysis of the effect of man's development of the river in the study area in Chapters 3 and 4 set the stage for the detailed analysis of dredging and disposal operations in Chapter 5.

Chapter 2

THE GEOLOGIC SCENE

2.1 Preglacial Drainage Patterns

The basin of the Mississippi River above Cairo, Illinois is located predominantly in the Central Lowlands physiographic province. The southern portion of the basin skirts the Ozark Plateau and enters the Mississippi Embayment of the Coastal Plains province just above Cairo (Figure 1). In general, the basin is underlain by the igneous-metamorphic rock complex of the southern part of the Canadian Shield. These rocks, Precambrian in age, extend southward beneath the overlying rocks and emerge again in the Ozark Plateau, forming a basin in which a thick sequence of sediment occurs. Sandstone, limestone, and shale were deposited in the basin throughout Paleozoic time (Upper Mississippi River Basin Coordinating Committee, 1972).

During subsequent Mesozoic and Cenozoic time the area was above sea level and erosion took place. While early investigators surmised that the valley of the Mississippi River above Cairo was formed following the deposition of glacial drift, it is now generally believed that the present drainage lines were formed by preglacial erosion of the ancient uplifted surface, producing a well integrated drainage system in the basin prior to the Pleistocene (Ruby, 1952; Thornbury, 1965).

The preglacial Central Lowlands drained north to Hudson Bay, east to the St. Lawrence, and south to the Gulf of Mexico, with a major drainage divide below today's Great Lakes (Figure 2). Essentially two major ancient drainage systems existed in the study area. The Rock and Teays Rivers constituted one system which joined near

Hennepin, Illinois to form the ancient Mississippi River and then flowed down the Illinois valley. The second system feeding the Iowa, rose in southern Minnesota and flowed across Iowa to Muscatine, then turned south along the present Mississippi valley to join the eastern drainage at the mouth of the present Illinois River (Thornbury, 1965).

2.2 The Influence of Glaciation

During the Pleistocene, four successive continental ice sheets covered much of the upper Mississippi basin. The glaciers left a varying thickness of deposits, or drift, ranging from a thin veneer to a layer several hundred feet thick. In addition, much of the surface of the study area was mantled with a layer of loess, windblown silt, locally as thick as 300 feet, derived from glacial deposits prior to the development of extensive vegetation (Upper Mississippi River Basin Coordinating Committee, 1972).

The preglacial drainage pattern was altered during the Pleistocene. A significant part of the Hudson Bay drainage was diverted to the Gulf of Mexico and the ancient rivers were repeatedly forced out of their valleys. The drainage pattern as it existed at the close of the Nebraskan glaciation (Aftonian interglacial) is shown in Figure 3a. The Kansan continental glaciation (Figure 3b) strongly influenced this pattern. The Iowa River system was diverted to the east by the ice to join the Rock-Teays-Mississippi River system which occupied the Illinois valley. This large glacial river cut a deep bed rock valley, now abandoned, between Fulton and Hennepin, Illinois. At the same time the ancestral Ohio River was forced to the south (Frye et al., 1965).

Following the Kansan glaciation the drainage pattern of the Aftonian interglacial was re-established, with the ancient Mississippi River occupying the Illinois valley and the ancestral Iowa River re-occupying the present Mississippi River valley (Figure 3c). Then, during the Illinoian glaciation (Figure 3d), the ice sheet advanced from the northeast and forced the ancient Mississippi west to form a temporary channel across Iowa. A lobe of the ice sheet partly blocked the Mississippi valley at St. Louis, causing deposition upstream.

Again, during the intervening Sangamonian interglacial (Figure 3e) the Mississippi reoccupied the Illinois valley and the Iowa drained through the present Mississippi valley. The final advance of the Wisconsin ice (Figure 3f) forced the Mississippi River into its present valley, and the Illinois River, now draining a much reduced area, occupied the valley formed by the ancient Mississippi.

During the retreat of the ice following the Wisconsin glaciation (Figure 3g) major floods such as the Kankakee Flood moved through the Illinois valley as ice dams failed and glacial lakes drained in the Chicago area. Since drainage of glacial melt water to the north and east was still blocked by ice, tremendous flows were carried out of the region by the Mississippi River drainage system. Glacial melt water collected to the north in a great basin forming Lake Agassiz, which covered much of the present areas of the Dakotas, Minnesota, Manitoba, and Saskatchewan. The Minnesota and Mississippi Rivers served as this lake's outlet and carried large volumes of water with relatively small sediment loads for the discharge involved, giving the water great erosive capability. As a result, the valley of the Mississippi was cut to a level at least 50 to 100 feet below the modern floodplain and enlarged far beyond the needs of the present river. With further retreat of the glaciers, drainage to the north and east was re-established. The volume and velocity of the melt water decreased and river valleys in the region were partially refilled by glacial outwash sediments consisting largely of sand and sandy gravel (Rubey, 1952).

Drilling logs from borings within the floodplain confirm the existence of a deeply buried bedrock valley underlying these alluvial deposits. Valley cross sections along the Middle Mississippi (Figure 4) reveal that the surface of the present alluvial plain is generally 80 to 150 feet above the bedrock valley floor. Drilling logs also verify the existence of progressively coarser granular material with depth (Degenhardt, 1973). This alluvial sequence, with its upward decrease in particle size provides proof of a gradual reduction in carrying capacity of the stream (Fisk, 1947).

As the ice front moved further north and sediment loads decreased, the river incised the alluvial valley floor to depths of 50 to 75 feet, leaving terrace and terrace remnants on both the master stream and tributaries. For example, on the Chippewa River (Figure 1) terraces up to 100 feet high are located adjacent to the present river flood-plain, providing a nearly inexhaustible source of coarse sediments (Corps of Engineers, St. Paul, 1974). Along the Mississippi, valley widening and floodplain development through bluff recession occurred during the late Pleistocene and has continued through the Recent to produce a river valley 1.5 to 5 miles in width (Rubey, 1952).

The close of the Wisconsin glaciation saw the establishment of the major features of today's drainage pattern in most of the study area

(Figure 3h). About 5000 years ago when sea level had stabilized near its present elevation, the Middle Mississippi River entered the Coastal Plains province at Cap Girardeau as a braided stream flowing to the west of the Commerce hills (Figure 6). Bluff cutting, probably assisted by local faulting, resulted in diversion of the Mississippi River to the east through Thebes Gap at progressively lower stages. About 2000 years ago the diversion was complete, and the Middle Mississippi completely abandoned its western braided course and joined the Ohio River at Cairo (Fisk, 1947).

2.3 The Influence of Tributaries

While the ancestral Mississippi River was cutting the preglacial surface, then trenching and widening its valley, a complex system of tributaries developed, and in turn influenced the character of the master stream. Larger tributaries such as the Minnesota, Iowa and Salt Rivers inherited part or all of their courses from the drainage pattern of the preglacial surface, but most of the smaller streams probably developed through headward erosion along fault lines (Rubey, 1952).

Tributary streams are sensitive to changes in base level produced by erosion or deposition in the master stream. Down cutting of the main stream lowers tributary base levels, increases gradients, and initiates headward erosion in a tributary. Conversely, a wave of alluviation moving through the main channel will gradually spread into tributary valleys as base levels are raised. The trenching of the Mississippi River into glacial outwash deposits produced by the Wisconsin glaciation induced head cutting in tributary valleys. The process of headward erosion establishes steeper gradients in the tributary streams than exists in the master channel. The tributaries, then, transport coarser sediments than the main stream can move, and deposition at the tributary confluence follows. The resulting fan-like deposit can influence both the position and the gradient of the main stream over extended reaches.

The most striking example of tributary influence in the study area is produced by the Chippewa River which enters the Mississippi above Wabasha, Minnesota (Figure 1). The gradient of the Chippewa River between the confluence and Eau Claire, Wisconsin is about two times greater than the gradient of the Mississippi in the region. By virtue of its steeper gradient and higher velocity the Chippewa has a greater sediment transport capacity than the Mississippi into which it carries several hundred thousand cubic yards of coarse material each year. While the ancient Mississippi had a discharge and velocity great enough to transport the Chippewa sediments, as drainage from Lake Agassiz ebbed the sediment carrying capacity of the Mississippi also declined. As a result an extensive alluvial fan formed at the mouth of the Chippewa, ponding the Mississippi and forming Lake Pepin (Zumberge, 1952).

2.4 Structural Features of the Study Area

Beneath the layer of glacial drift the surface of the bedrock in the Upper and Middle Mississippi River basin is relatively flat with some broad undulations of low relief. The geologic structure of the bedrock is largely determined by a major uplifted area (The Laurentian Uplift) to the north, from which the bedrock formations dip gently southward. Within the basin minor structures of broad domes, low anti-clinal arches, and broad, saucer-like basins modify the prevailing structure and dip of the bedrock. There are three principle examples of these features: the Illinois basin, a large spoonlike depression which modifies the general dip of the bedrock in the region; the Wisconsin dome (uplift) which covers the greater part of Wisconsin; and the Ozark Uplift which exposes Precambrian rocks and modifies the geology at the southwestern edge of the Mississippi basin (Upper Mississippi River Basin Coordinating Committee, 1972).

The Mississippi Embayment and the eastern and southern borders of the Ozark uplift are active seismic zones. The town of New Madrid, Missouri, located southwest of Cairo at the upper end of the Mississippi Embayment, was the site of the New Madrid earthquakes of 1811 and 1812, which produced some of the most intense shocks ever experienced in the continental United States (Corps of Engineers, St. Louis, 1974). As a result the Mississippi valley from New Madrid north to St. Louis is one of the most active earthquake regions in the United States east of California (Rubey, 1952).

A major geologic structure along the western edge of the Illinois basin, the Cap au Gris Fault zone, crosses the Mississippi valley just south of Cap au Gris, Missouri (Figure 5). The vertical displacement along this feature is about 1100 feet and the depth to bedrock is about 25 feet. However, Rubey (1952) concludes that there has been no recent movement along this fault zone, since the Pleistocene terraces are not warped or displaced as they cross the structure. Apparently, the modern Mississippi River is not being modified by external structural influences. A similar conclusion can be supported relative to the Middle Mississippi. Thus, the character of the modern Mississippi is primarily a function of the water and sediment transported through its channel, and geomorphic change is not the result of tectonic activity.

Although bedrock is covered by as much as 150 feet of alluvium in the Mississippi valley above Cairo, there are several regions where bedrock exposure has had an influence on river morphology. On the Upper Mississippi these include the exposure at St. Anthony Falls in the Minneapolis-St. Paul region (Figure 1) where the river has incised through glacial drift and deeply into the bedrock to form gorges with vertical exposures of bedrock (Upper Mississippi River Basin Coordinating Committee, 1972). There is also regional exposure of bedrock in the vicinity of Rock Island, Illinois and Keokuk, Iowa (Figure 1).

On the Middle Mississippi in the vicinity of St. Louis a ledge of resistant sedimentary rock at Chain of Rocks (Figure 6) created a hazard to navigation and has been bypassed by a canal and navigation lock. In addition, the natural diversions of the river at Grand Tower and Thebes Gap (Figure 6) have produced regions where a portion of the river flows on bedrock (Degenhardt, 1973). In addition to exercising a vertical control at various points along the river and creating navigation difficulties, bedrock exposure has exercised a lateral control of river position. The morphology of reaches downstream from such natural constrictions as Thebes Gap is strongly influenced by this lateral control.

2.5 <u>Summary</u>

The major features of the drainage pattern in the study area were established prior to the Pleistocene glaciation; however, four successive continental ice sheets during the Pleistocene produced significant modifications. The last glacial epoch, the Wisconsin, established the Mississippi River in its present course. Drainage of large volumes of sediment-free water from glacial lakes scoured the river valleys far below present floodplain levels. The melting of the Wisconsin ice sheets uncovered drainage paths to the north through Hudson Bay, and as a result, discharge to the south through the Mississippi decreased. These smaller flows were unable to transport glacial sediments on the flat slopes established by earlier torrents and a wave of alluviation partially filled the main stream and tributary valleys. Finally, as the ice front moved further north, the reduction in sediment load enabled the river to incise the alluvial valley floor, leaving terraces and terrace remnants along the master stream and tributaries. Subsequent valley widening and floodplain development occurred in post-glacial time.

The influence of tributaries in establishing the position and gradient of the Mississippi River within the river valley, is illustrated by the response of the main stream to the deposits of the Chippewa River. Although the Mississippi in the study area crosses several major fault zones and traverses a seismically active region, the position and character of the modern river do not appear to be strongly influenced by either faulting or structural movement. However, at several locations bedrock exposure does constitute a vertical or lateral control.

The morphology of the modern Mississippi in the study area, then, is primarily the result of Pleistocene history modified by tributary influence and by the varying amounts of water and sediment delivered to the channel in post-glacial time.

Chapter 3

THE GEOMORPHIC PERSPECTIVE

3.1 The Natural River

3.1.1 Definition

Man's improvement of the Mississippi River above Cairo for navigation and flood control has been underway for almost 150 years. In 1824 Congress authorized the Corps of Engineers to conduct snagging operations, that is, removal of sunken debris hazardous to navigation. On the Middle Mississippi from 1824-1880 some low-level levees were constructed along the river banks by private land owners to prevent local flooding of the floodplain. In 1879 passage of the Illinois State Drainage and Levee Act established organized levee districts to accomplish the needed works with the aid of state funds; however, levee construction was not intensive until 1907 (Simons et al., 1974).

In 1881 a comprehensive plan for regulation of the Middle Mississippi was approved by Congress. This plan called for the continuous improvement of the navigation channel by reducing the width of the river with wing dams (dikes) and bankline revetment. In about this same time period the first dikes and revetment were constructed in the Upper Mississippi.

The earliest dredging on the Upper Mississippi was conducted with devices that employed stirring or scraping techniques to increase navigation depths. In 1867 $96,000 was appropriated for the construction and operation of two scrapers on the Mississippi between St. Paul and the mouth of the Illinois. Although the operation of scraping devices was deemed a success, at best they increased depths over short bars by only 12 to 18 inches. It was not until 1895 that the first hydraulic dredges were used for experimental dredging operations on the Mississippi (Ockerson, 1898).

This brief summary of early improvements on the river indicates that in general, the river was unaffected by development until the late 1890's. Prior to this time man's efforts at development were either too localized or ineffective to have altered the natural regimen of the river significantly. Therefore, the nineteenth century river will be considered as "natural" and the twentieth century river as "developed". This dichotomy is strongly supported by a comparison of river morphology circa 1820 as revealed by township plats, which provide the earliest accurate record of river position, and the 1891 hydrographic survey, which is the earliest complete hydrographic survey of the upper river. This comparison reveals that the natural river of 1820 was not altered appreciably through the 1891 time period.

3.1.2 The River

In establishing conditions of the natural river, journals of early explorers provide a detailed, if non-technical, description. For example, on reaching the junction of the Wisconsin and the Mississippi Rivers (Figure 1) on 17 June 1673, Joliet and Marquette found "before them a wide and rapid current...by the foot of lofty heights wrapped thick in forest..." Turning south they journeyed "through a solitude unrelieved by the faintest trace of man." At the mouth of the Missouri they found that "a torrent of yellow mud rushed furiously athwart the calm blue current of the Mississippi, boiling and surging and sweeping in its course logs, branches, and uprooted trees" (Twain, 1968).

Henry Shreve was one of the first to recognize the commercial potential of the river as a transport route; however, travel on the early river was made extremely hazardous by the thousands of snags formed when uprooted trees became imbedded in the channel or stacked on islands and bars. Although boat owners and settlers pleaded with Congress for action, it was considered impossible to remove the snags. "Trees, whose roots had dug deep into the stream bottom and became planted, were packed down by tons of silt that had caught against them; those piled against bars were snarled together in great masses. Only a race of giants could remove them" (Dorsey, 1941).

Attempts at removal of this feature of the natural river proved inadequate until Shreve developed the steam powered snag boat, Heliopolis, in 1829. Shreve completed the first successful snag removal operation at Plum Point, one of the worst timber clogged reaches on the river. He reported his results to the Chief of Engineers in Washington: "There I made the first attempt to remove snags with the boat and am prout (sic) to say that the performance far exceeded my most sanguine expectations. In eleven hours that whole forest of formidable snags, so long the terror of Boatmen (many of which were six feet in diameter) were effectually removed." By the end of 1830, the age old drowned forests had vanished from the river between the mouth of the Missouri to Bayou Sara, just north of Baton Rouge (Dorsey, 1941).

Humphreys and Abbot (1876) in their classic "Report upon the Physics and Hydraulics of the Mississippi River" provide the earliest rigorous description of the natural river, and summarize observations of earlier exploration of the upper river. Allen and Schoolcraft visited the source of the Mississippi, Lake Itasca (Figure 1), in 1832 and described the outlet of the lake as "10 or 12 feet broad, with an apparent depth of 12 to 18 inches." Below St. Anthony Falls: "the river expands into Lake Pepin, which is 2 or 3 miles broad and 27 miles long...From Lake Pepin to the junction of the Missouri, the Mississippi is characterized by almost innumerable wooded islands. The main volume of the stream is confined to one channel, but branches from it ramify in all directions, forming sloughs...and making its water-course, with enclosed islands seldom less than a mile in width..."

Below the mouth of the Missouri (Figure 6) Humphreys and Abbot found that "the Mississippi river first assumes its characteristic appearance of a turbid and boiling torrent, immense in volume and force. From that point its waters pursue their devious course for 1300 miles, destroying banks and islands at one locality, reconstructing them at another...After passing the bottomlands near the mouth of the Missouri, the right (west) bank of the Mississippi is mainly composed of high limestone bluffs, which seldom recede more than a mile or two from the river until Cape Girardeau is reached...Commerce bluffs next border the river for a few miles...The left (east) bank of the Mississippi from the mouth of the Missouri to the mouth of the Kaskaskia consists of a strip of lowland called the American bottom, which is subject to overflow in the highest floods. Thence to Commerce the bank is formed of bluffs like those on the opposite side of the river" (Humphreys and Abbot, 1876).

In regard to the character of the bed of the Mississippi and formation and stability of islands and chute channels under natural conditions, Humphreys and Abbot made several valuable observations. It was "evident to the eye at low water--that immense beds of pure silicious sand and fine gravel, entirely free from the muddy sedimentary matter with which the water is charged, exist in the channel-way. They are found below points, in island chutes, sometimes, though rarely, entirely across the bed, and, in general wherever the water moves with a current too rapid to deposit its sediment...Opposite caving bends, in eddies below islands, and at other points where for any cause the current becomes nearly dead, the sediment transported by the river-water is deposited, forming gently-sloping, sandy, mud banks, called willow battures (or, if on islands, towheads), from the growth of willows which soon makes its appearance upon them. This process of land-formation serves to fix a normal limit beyond which the river can not increase its width by caving..."

"Upon the islands the action of the Mississippi is not less striking than upon the banks. They are constantly forming, disappearing, or becoming connected with the main land by the filling up of their chutes. The process of formation and destruction is interesting. Drift-wood becomes lodged upon a sandbar. Deposition of sediment follows. A willow growth succeeds. In high water more deposition is caused by the resistance thus presented to the current...An island thus rises gradually to the level of high water, and sometimes even above it, sustaining a dense growth of cottonwoods, willows, etc. By a similar process the island becomes connected with the mainland; or, by a slight change of direction of the current, the underlying sandbar is washed away. The new made land caves into the river, and the island disappears" (Humphreys and Abbot, 1876).

One of the most complete engineering descriptions of the river while it could still be considered in the natural state was given by Ockerson in his milestone paper, "Dredges and Dredging on the Mississippi River" (1898). Ockerson noted that between St. Anthony Falls and the mouth of the Missouri the "banks are low, and the oscillation between high and low water rarely exceeds 25 ft. In the upper half of this reach the river is divided into a great many sloughs, which serve as high-water channels, but are often nearly or quite dry at low water. The water carries but little sediment; bank erosion is comparatively slight; for 21 miles it flows through a lake of slack water 30 ft deep (Lake Pepin); the flow in two places is interrupted by rapids where the bed of the stream is solid rock (Rock Island and Keokuk); in the upper portion, the navigable depth at low water sometimes gets down to 2 1/2 ft, and navigation is usually suspended during the winter season for a period of four months or more in consequence of the river being frozen. The low-water slope averages about 0.5 ft per mile. The low-water discharge is about 25,000 cu. ft per second. High water generally comes in May and June, and the low-water season usually begins about the first of September and lasts until navigation is closed by ice."

In describing the Middle Mississippi Ockerson notes, as did earlier writers, the abrupt change in character of the river below the Missouri, and also alludes to the effects of early contraction works. "This reach is the first to take up the enormous load of sediment put upon it by the Missouri River. Here permeable dikes are at their best, and immense deposits are easily induced where channel contraction is desirable to increase the depth. The banks are somewhat higher than those of the first reach (Upper Mississippi) and the effects of bank erosion are more noticeable. The extreme oscillation between high and low water near the upper portion of this reach is some 36 ft; the low-water averages 0.6 ft per mile: the low-water discharge is about 45,000 cu. ft per second, and the high-water discharge about 850,000 cu. ft per second. At low water the navigable depths on the bars often reach as low as 4 ft. Overflows are not very frequent, as a conjunction of the floods of the upper Mississippi and Missouri Rivers is necessary to produce an overflow stage. The high-water stages usually occur in May and June, and the low-water season begins early in September and continues into the winter months." Ockerson was also impressed by the dynamic character of the river, particularly in regard to the formation of sand bars which "are numerous, and crossings are consequently frequent, and their locations are constantly shifting" (Ockerson, 1898).

Tiefenbrun's summary (1963) adds some detail to the descriptions of the Middle Mississippi already presented. In its natural state the river was relatively wide and shallow, with the channel being divided by numerous bars and islands and their accompanying chutes and sloughs. At low water, channel depths on the Middle Mississippi varied from 3 to 10 feet over widths ranging from 125 to 2500 feet. At flood stage, depths varied from 21 to 60 feet and widths from 2500 to as much as 25,000 feet.

In the mid-1800's engineers were not the only individuals who recorded their observations of the upper river. Twain looked at the river with a poet's eye and observed on returning to Hannibal: "The extensive view up and down the river, and wide over the wooded expanses of Illinois is very beautiful...The eight hundred miles of river between St. Louis and St. Paul afford an unbroken succession of lovely pictures...The majestic bluffs that overlook the river...charm one with the grace and rariety of their forms and the soft beauty of their adornment. The steep verdant slope, whose base is at the water's edge, is topped by a lofty rampart of broken turreted rocks, which are exquisitely rich and mellow in color...And then you have the shining river, winding here and there and yonder, its sweep interrupted at intervals by clusters of wooded islands threaded by silver channels..." (Twain, 1968).

Just as Twains' impression of the upper river is in consonance with the character of the natural river as recorded by engineers, Dickens also conveyed the change in character of the river below the mouth of the Missouri. He perceived the Mississippi as "an enormous ditch, sometimes two or three miles wide, running liquid mud, six miles per hour: its strong and frothy current choked and obstructed everywhere by large logs and whole forest trees..." (Simons et al., 1974).

From a more practical point of view, Twain described the dynamic character of the Middle Mississippi and the resulting problems of the riverboat pilot. The river "changes its channel so constantly that the pilots used to always find it necessary to run down to Cairo to take a fresh look, when their boats were to lie in port for a week; that is, when the water was at a low state...When the river is very low, and one's steamboat is 'drawing all the water' there is in the channel--or a few inches more, as was often the case in the old times...we used to have to 'sound' a number of particularly bad places almost every trip..." (Twain, 1968). Another river pilot, I. H. Baldwin, noted that when the river had fallen to bankfull stage after the flood of 1844, it was difficult to chart a navigable channel between St. Louis and the Ohio River (Degenhardt, 1973).

3.1.3 Summary

The picture of the natural river that emerges is one that strongly reflects the influence of Pleistocene glaciation, and its aftermath, on the character of the river. The sequence of trenching of the preglacial sedimentary surface, then filling with glacial out wash, followed by incision of the alluvial deposits, produced the natural river of the 19th century. The river is truly alluvial in character, that is, it flows essentially in cohesive or noncohesive materials that have been or can be transported by the stream. In general the river follows a winding course between low natural levees in a wide floodplain bordered by the high bluffs of sedimentary rock described by Twain.

As early descriptions indicate, the Mississippi above the Missouri can be considered a clearwater stream; however, the Missouri delivers so much sediment that below the junction the Middle Mississippi must be classed as a heavy sediment carrier. As Mack (1970) points out, the broad classification of the Upper Mississippi as a clearwater stream is relative. In fact the upper river does transport over 20×10^6 tons of sedimentary material annually. This quantity of material is sufficient to be an important factor in maintaining navigable depths on the upper river.

The phenomena of growth and destruction of islands, chute channels, and sloughs within the natural river has elicited comment by almost every observer of the early river. As noted by Winkley et al., (1972), in an alluvial river it is the rule rather than the exception that banks will erode, sediments will be deposited, and floodplains, islands, chutes, and side channels will change with time. This is clearly evident on the Mississippi above Cairo. The natural growth and decay of islands, side channels, and sloughs is of central importance to the dredging and disposal problem, and accordingly, is analyzed in detail in following sections.

Although morphologic change within the channel itself is the norm, subsequent analysis shows that in terms of position within the flood-plain, both the Middle and Upper Mississippi are relatively stable. This is particularly true if one gages stability in comparison to the Lower Mississippi where Fisk's maps (1947) reveal large scale lateral shifting of the river's position, downstream migration of the meander pattern, and numerous natural cutoffs of meander loops (Figure 7). In fact, the relative stability of the upper river permits vegetation growth to the water line, limits the scars due to bank failure, and produces some of the finest river scenery in the world.

3.2 River Mechanics and Morphology

3.2.1 General Concepts

The dynamic character of the Mississippi River was a recurring element in the descriptions of the natural river presented in Section 3.1. Rivers are indeed, dynamic systems and respond to changing climatic, hydrologic, geomorphic, and geologic conditions. As is apparent from the geomorphic analysis of subsequent sections, rivers are also extremely sensitive to change induced by man's development. This section provides a brief summary of definitions and general concepts necessary to an understanding of the mechanics of river systems. In addition, several aspects of river mechanics and morphology that bear directly on the analysis of dredging and disposal effects are examined in detail. These include: the crossing and pool sequence in both meandering and "straight" reaches, the island and side channel configuration of divided reaches, and pertinent sediment transport concepts.

3.2.1.1 Variables Affecting Alluvial Channels. Because of the large number of interrelated variables that can respond simultaneously to natural or imposed changes in a river system, the mechanics of flow in alluvial channels is complex. In addition, river form, channel geometry, islands, bars, and bed roughness are all subject to a continual process of evolution as the parameters of water and sediment discharge change.

Lane (1957) indicates that the most important variables affecting alluvial channels are: stream discharge, slope, sediment load, resistance of banks and bed to movement by flowing water, vegetation, temperature, geology, and the works of man. These eight are not the only factors involved, but Lane postulated that they were the major factors. In addition, these variables are not all independent. For example, the interrelation among slope, sediment load, and resistance is particularly close and complex.

Simons has given a more detailed analysis of the variables affecting alluvial channel geometry and bed roughness (Shen, 1971b), and concludes that the nature of these variables "is such that, unlike rigid boundary hydraulics problems, it is not possible to isolate and study the role of an individual variable." For example if one attempts to evaluate the effect of increasing channel depth on average velocity, additional related variables respond to the changing depth. Thus, not only will velocity respond to a change in depth, but also the form of bed roughness, the shape of the cross section, the quantity of sediment discharge, and the position and shape of alternate, middle, and point bars can be expected to change.

The list of variables that influence alluvial channel flow should include:

$$\phi[V, D, S, \rho, \mu, g, d, \sigma, \rho_s, S_p, S_R, S_C, f_s, C_T, C_F, \omega] = 0 \quad (1)$$

in which

V = velocity

D = depth

S = slope of energy grade line

ρ = density of water-sediment mixture

μ = apparent viscosity of water-sediment mixture

g = gravitational constant

d = representative fall diameter of the bed material

σ = gradation of bed material

ρ_s = density of sediment

S_p = shape factor of the particles

S_R = shape factor of the reach of the stream

S_C = shape factor of the cross section of the stream

f_s = seepage force in the bed of the stream

C_T = concentration of bed-material discharge

C_F = fine material concentration

ω = particle terminal fall velocity

In general, river problems are confined to flow over beds consisting of quartz particles with constant ρ_s. Also the value of g is usually taken as constant.

Applying techniques of dimensional analysis to this list of variables, with V, D, and ρ selected as repeating variables, yields:

$$\phi_1[S, \frac{VD\rho}{\mu}, \frac{V}{\sqrt{gD}}, \frac{d}{D}, \sigma, \frac{\rho_s}{\rho}, S_p, S_R, S_C, \frac{f_s}{\rho V^2}, C_T, C_F, \frac{\omega}{V}] = 0 \qquad (2)$$

Equation (2) provides a list of nondimensional parameters which are important in a study of alluvial channel characteristics. These include the Froude number ($V/\sqrt{gD}$), the Reynolds number ($VD\rho/\mu$) and a relative roughness parameter (d/D).

The problems presented by the interdependency of these variables become apparent when an attempt is made to differentiate between dependent and independent variables. Consideration of the slope of the energy grade line of an alluvial stream illustrates the changing role of a variable and the difficulty of selecting independent variables, particularly in field studies. If a stream is in equilibrium with its environment, slope is an independent variable. The stream is thought of as "graded" or "poised" since the average slope over a period of years has adjusted so that the flow is capable of transporting only the amount of sediment supplied at the upper end by tributaries, and the stream neither aggrades nor degrades its channel. If for some reason a tributary or upstream reach supplies a larger or smaller quantity of sediment than the stream is capable of carrying, the slope changes in response to the amount of sediment supplied. Slope, in this case, is a dependent variable.

3.2.1.2 Channel Patterns. Rivers can be classified broadly in terms of channel pattern, that is, the configuration of the river as viewed on a map or from the air. Patterns include straight, meandering, braided, or some combination of these (Figure 8).

A straight channel can be defined as one that does not follow a sinuous course. Leopold and Wolman (1957) have pointed out that truly straight channels are rare in nature. Although a stream may have relatively straight banks, the thalweg, or path of greatest depths along the channel, is usually sinuous (Figure 8b). As a result, there is no simple distinction between straight and meandering channels.

The sinuosity of a river, the ratio between thalweg length to down valley distance is most often used to distinguish between straight and meandering channels. Sinuosity varies from a value of unity to a value of three or more. Leopold, Wolman, and Miller (1964) took a sinuosity of 1.5 as the division between meandering and straight channels. It should be noted that in a straight reach with a sinuous thalweg developed between alternate bars (Figure 8b) a sequence of shallow crossings and deep pools is established along the channel.

Reaches of a river that are relatively straight over a long distance are generally classed as unstable, as are divided flow reaches, and those in which bends are migrating rapidly. Long straight reaches can be created by natural or man-made cutoff of meander loops where long reaches of sinuous meandering channels with relatively flat slopes are converted to shorter reaches with much steeper slopes. Straight reaches can also be man-induced by placing of contraction works such as dikes and revetment to reduce or control sinuosity. As noted, even where the channel is straight it is normal for the thalweg to wander back and forth from one bank to the other. Opposite the point of greatest depth there is usually a bar or accumulation of sediment along the bank, and these bars tend to alternate from one side of the channel to the other (Figure 9). At low stages, then, the thalweg in a straight reach tends to meander within the high-water channel, forming short pools and relatively long and shallow crossings generally unsuitable for navigation. The alternate bars control channel pattern and, thus, their stability determines the stability of the reach.

A braided river is generally wide with poorly defined and unstable banks, and is characterized by a steep, shallow course with multiple channel divisions around alluvial islands (Figure 8a). Braiding was studied by Leopold and Wolman (1957) in a laboratory flume. They concluded that braiding is one of many patterns which can maintain quasi-equilibrium among the variables of discharge, sediment load, and transporting ability. Lane (1957) concluded that, generally, the two primary causes that may be responsible for the braided condition are: (1) overloading, that is, the stream may be supplied with more sediment than it can carry resulting in deposition of part of the load, and (2) steep slopes, which produce a wide shallow channel where bars and islands form readily.

Either of these factors alone, or both in concert, could be responsible for a braided pattern. If the channel is overloaded with sediment, deposition occurs, the bed aggrades, and the slope of the channel increases in an effort to maintain a graded condition. As the channel steepens, the velocity increases, multiple channels develop and cause the overall channel system to widen. The multiple channels, which form when bars of sediment accumulate within the main channel, are generally unstable and change position with both time and stage.

A meandering channel is one that consists of alternating bends, giving an S-shape appearance to the plan view of the river (Figure 8c). More precisely, Lane (1957) concluded that a meandering stream is one whose channel alignment consists principally of pronounced bends, the shapes of which have not been determined predominantly by the varying nature of the terrain through which the channel passes. The meandering river consists of a series of deep pools in the bends and shallow crossings in the short straight reach connecting the bends. The thalweg flows from a pool through a crossing to the next pool forming the typical S-curve of a single meander loop.

As shown schematically in Figure 8, the pools tend to be somewhat triangular in section with point bars located on the inside of the bend. In the crossing the channel tends to be more rectangular, widths are greater and depths are relatively shallow.

The phenomenon of meandering has engendered great interest as evidenced by the number of papers on the subject to be found in the literature. Both statistical arguments and concepts of dynamic instability have been advanced as explanations (Khan, 1971); however, most arguments invoke the existence of secondary currents in the bendway as a prime contributor to the meandering process.

Secondary currents occur in bends of pipes and open channels as a result of the difference of centrifugal forces acting on flow lines of different velocities. The existence of secondary currents in bends and their consequent effect on the movement of bed material support the contention that they play a significant role in the growth and migration of a meander (Khan, 1971). Because of the importance of the pool and crossing sequence to the dredging process, the mechanics of flow in a meander loop is examined in detail subsequently.

Leopold and Wolman (1957) pointed out that because of the physical characteristics of straight, braided, and meandering streams, all natural channel patterns intergrade. Although braiding and meandering patterns are strikingly different, they actually represent extremes in a continuum of channel patterns. On the assumption that the pattern of a stream is determined by the interaction of numerous variables whose range in nature is continuous, one should not be surprised at the existence of a complete range of channel patterns. A given reach of a river, then, may exhibit both braiding and meandering, and alteration of the controlling parameters in a reach can change the character of a given stream from meandering to braided or vice versa.

A number of studies have quantified this concept of a continuum of channel patterns. Khan (1971) related sinuosity, slope, and channel pattern (Figure 10). Any natural or artificial change which alters channel slope such as the cutoff of a meander loop, can result in modifications to the existing river pattern. A cutoff in a meandering channel shortens channel length, increases slope, and tends to move the plotting position of the river to the right on Figure 10. This indicates a tendency to evolve from a relatively tranquil, easy to control meandering pattern to a braided pattern that varies rapidly with time, has high velocities, is subdivided by sandbars, and carries relatively large quantities of sediment. Conversely, a slight decrease in slope could change an unstable braided river into a more stable meandering pattern.

Lane (1957) investigated the relationship among slope, discharge and channel pattern in meandering and braided streams, and observed that an equation of the form

$$SQ^{1/4} = K \tag{3}$$

fits a large amount of data from meandering sand streams. Figure 11 summarizes Lane's plots and shows that when

$$SQ^{1/4} \leq .0017 \tag{4}$$

a sand bed channel will tend toward a meandering pattern. Similarly, when

$$SQ^{1/4} \geq .01 \tag{5}$$

a river tends toward a braided pattern. Slopes for these two extremes differ by a factor of almost 6. The region between these values of $SQ^{1/4}$ can be considered a transitional range where streams are classified as intermediate. Many rivers of the United States fall in this intermediate category. If a river is meandering, but with a discharge and slope that borders on transitional, a relatively small increase in channel slope could initiate a tendency toward a transitional or braided character.

With reference to Figure 11, the Upper Mississippi (points 120 and 121) plots in the meander zone, and the Middle Mississippi (points 19 and 20) plots just above the meander zone as an intermediate stream. Other river reaches of interest to this study are also shown on Figure 11. Although there is a definite meandering aspect to the Upper Mississippi, descriptions of the natural river's divided channels and numerous alluvial islands also support the contention that at least portions of the upper river have a braided character. However, the

Upper Mississippi does not have the steep slopes generally associated with braiding.

Lane (1957) recognizes the braided character of some reaches in the Upper Mississippi (Figure 12) and notes that the conditions producing braiding are unusual. Braiding on the Upper Mississippi is of the overloading type and is closely related to the unique glacial history of the basin. The channel which was excavated by clear water torrents from glacial lakes was on too flat a slope to transport glacial sediments as the ice front retreated to the north and water discharge diminished. A great filling of the stream valley began and continues today. This filling, braided channel produced the multiplicity of interlacing channels and islands to be found near Lansing, Iowa (Figure 12) and in many other reaches between St. Paul and St. Louis.

According to Lane, the slope of the Mississippi is still too flat to move all of the sediment brought to it, and filling will continue to steepen the slope until the required transport capacity is obtained. Aggrading of the valley proceeds slowly, as the entire width of the valley must be filled, and most of the deposition takes place in the quiet depositional environment of ponds, lakes, and secondary channels. The main channel should tend to enlarge as side channels are filled. When aggradation has progressed to the stage where the river can carry the entire sediment load supplied, it is probable that there will be a main channel largely free of islands, sloughs, and secondary channels (Lane, 1957).

3.2.1.3 The Longitudinal Profile. The longitudinal profile of a stream shows its slope, or gradient. It is a visual representation of the ratio of the fall of a stream to its length over a given reach (Morisawa, 1968). Since a river channel or river system is generally steepest in its upper regions, most river profiles are concave upward. As with other channel characteristics, shape of the profile is undoubtedly the result of a number of interdependent factors. It represents a balance between the transport capacity of the stream and the size and quantity of the sediment load supplied.

Shulits (1941) provided an equation for the concave horizontal profile in terms of distance along the stream:

$$S_x = S_o e^{-\alpha x} \tag{6}$$

where: S_x = the slope at any station a distance x downstream of a reference station

S_o = the slope at a reference station

α = a coefficient of abrasion

As implied by the definition of the parameter, α, Shulits assumed that grain size decreases in a downstream direction, a fact confirmed by field observations on many rivers, to include the Mississippi (Figure 13). Transport processes alter the size of sediment particles by abrasion and hydraulic sorting. Abrasion is the reduction in size of particles by mechanical action such as grinding, impact, and rubbing, while hydraulic sorting is the result of differential transport of particles of different sizes. For sedimentary particles of similar shape, roughness, and specific gravity, the end result of these processes is the observed reduction of bed material size along the direction of transport (Rana et al., 1973). The change in particle size with distance downstream can be expressed as

$$d_{50x} = d_{50o}\, e^{-\beta x} \tag{7}$$

where: d_{50x} = median size of bed material at distance x downstream of a reference station

d_{50o} = median size of bed material at the reference station

β = a wear or sorting coefficient

This relationship, which plots as a straight line on semi-logarithmic coordinates, is shown for the Lower Mississippi in Figure 13b.

The longitudinal profile of an alluvial river is not static. It must adjust to continually changing input conditions of water and sediment discharge. Although adjustment to changing input conditions is realized by modification of channel geometry, roughness, and other parameters including gradient, a simplified model of the adjustment process can be obtained if it is assumed that a stream adjusts to transport its load by modifying only its gradient.

If a river is unable to move its load below a given point on the profile, as with the post-glacial Upper Mississippi, it will increase its gradient at that point by deposition (Figure 14a). This builds up the channel bed, causing an increased slope below the point and thus an increased ability to transport. At the same time deposition results in a decrease in gradient and transport capacity above the point, and a wave of aggradation moves upstream.

If a stream develops an excess ability to transport and can carry more sediment than is supplied in a given reach, it will lower its gradient (Figure 14b). It does this by scouring its channel at the point of excess capacity. This decreases the slope and transport capacity below the point but steepens the slope above the point. A wave of erosion or headcutting will then move upstream.

3.2.1.4 Qualitative Response of River Systems. The response of channel pattern and longitudinal gradient to variation in selected parameters has been discussed in previous sections. In more general terms, Lane (1955b) studied the changes in river morphology in response to varying water and sediment discharge. Similarly, Leopold and Maddock (1953), Schumm (1971), and Santos and Simons (1972) have investigated channel response to natural and imposed changes. These studies support the following general relationships:

(1) Depth of flow (D) is directly proportional to water discharge (Q) and inversely proportional to sediment discharge (Q_s).

(2) Channel width (W) is directly proportional to both water discharge (Q) and sediment discharge (Q_s).

(3) Channel shape, expressed as width to depth (W/D) ratio is directly related to sediment discharge (Q_s).

(4) Channel slope (S) is inversely proportional to water discharge (Q) and directly proportional to both sediment discharge (Q_s) and grain size (d_{50}).

(5) Sinuosity (s) is directly proportional to valley slope (S) and inversely proportional to sediment discharge (Q_s).

(6) Transport of bed material (Q_s) is directly related to stream power ($\tau_o V$) and concentration of fine material (C_F), and inversely related to the fall diameter of the bed material (d_{50}).

A very useful relation for predicting system response can be developed by establishing a proportionality between bed material transport and several related parameters.

$$Q_s \sim \frac{(\tau_o V) W C_F}{d_{50}} \tag{8}$$

where: τ_o = bed shear

V = cross-sectional average velocity

C_F = concentration of fine material load

Equation (8) can be modified by substituting γDS for τ_o, and:

$$Q = AV = WDV \tag{9}$$

from continuity, yielding

$$Q_s \sim \frac{(\gamma DS)WV}{d_{50}/C_F} = \frac{\gamma QS}{d_{50}/C_F} \tag{10}$$

If specific weight, γ, is assumed constant and the concentration of fine material, C_F, is incorporated in the fall diameter, this relation can be expressed simply as

$$QS \sim Q_s d_{50} \tag{11}$$

Equation (11) is essentially the relation proposed by Lane (1955a), except fall diameter, which includes the effect of temperature on transport, has been substituted for the physical median diameter used by Lane.

Equation (11) is most useful for qualitative prediction of channel response to natural or imposed changes in a river system. To use a classic example, consider the downstream response of a river to the construction of a dam (Figure 15). Aggradation in the reservoir upstream of the dam will result in relatively clear water being released downstream of the dam, that is, Q_s will be reduced to Q_s^- downstream. Assuming fall diameter and water discharge remain constant, slope must decrease downstream of the dam to balance the proportionality of equation (11)

$$Q_s^- \, d_{50}^o \sim Q^o S^- \tag{12}$$

In Figure 15 the original channel gradient between the dam and a downstream geologic control (line CA) will be reduced to a new gradient (line C'A) through gradual degradation below the dam. With time, of course, the pool behind the dam will fill and sediment would again be available to the downstream reach. Then, except for local scour, the gradient C'A would increase to the original gradient CA to transport the increase in sediment load. Upstream, the gradient would eventually parallel the original gradient, offset by the height of the dam. Thus, dams with small storage capacity may induce scour and then deposition over a relatively short time period.

The foregoing general concepts are important to an understanding of any problem of river mechanics and river system behavior. With these established, the crossing and pool sequence in meandering and straight reaches, the island and chute channel configuration of divided reaches, and certain sediment transport concepts are examined in detail. These three aspects of river mechanics are of central importance to the dredging and disposal problem.

3.2.2 Pools and Crossings

The crossing and pool sequence is common to both meandering reaches (Figure 8c) and straight reaches with a thalweg that meanders through alternate bars (Figure 8b). Friedkin (1945) established this basic characteristic of meandering streams in his classic investigation of meandering of alluvial rivers. Both small-scale model rivers and natural streams are deep along the concave bank of bends and shallow in the tangents between bends (Figure 8). Consequently, the thalweg profile exhibits a series of deeps (pools) separated by shoals (crossings or bars).

Cross sections in bends are triangular in shape (Figure 16c) with the deepest points located near the outer (concave) bank and shallow point bars located on the inner (convex) bank (Figure 16a). In the transition zone between bends, flow lines straighten and the cross section takes the form of a wide, shallow trough (Figure 16b) forming a saddle or bar which the thalweg must cross in moving from pool to pool. The crossing controls the least available depth through the reach for navigation at a given stage, and it is here, on the crossings, that dredging to obtain navigable depths is usually concentrated.

3.2.2.1 Hydraulics of Flow in Bends. The hydraulics of steady flow in a rigid boundary bend provides the basis for understanding the morphology of alluvial channel bendways. The change in flow direction around bends produces centrifugal forces which result in a higher elevation on the concave bank than on the convex bank. The resulting transverse slope can be evaluated using cylindrical coordinates as sketched in Figure 17. The differential pressure in the radial direction can be obtained by equating radial forces to the product of mass and radial acceleration:

$$[p - (p + \frac{\partial p}{\partial r}\,\delta r)]r\delta\theta\delta z = -(\rho\delta r\delta z r\delta\theta)\,\frac{V_\theta^2}{r}$$

$$\frac{1}{\rho}\frac{\partial p}{\partial r} = \frac{V_\theta^2}{r} \tag{13}$$

The total superelevation between outer and inner bank is

$$\Delta z = \frac{1}{\rho g}\int_{p_i}^{p_o} dp = \frac{1}{g}\int_{r_i}^{r_o} \frac{V_\theta^2}{r}\, dr \tag{14}$$

Assuming that radial and vertical velocities are small compared to the tangential velocities, $V_\theta \simeq V$; and that the pressure distribution in the bend is hydrostatic, $p = \gamma h$, yields:

$$\Delta z = \frac{1}{g} \int_{r_i}^{r_o} \frac{V^2}{r}\, dr \tag{15}$$

Equation (15) can be solved if the velocity distribution along the radius of the bend is known or assumed. For example, if V is the average velocity, Q/A, and r equals the radius to the center of the stream, r_c, then

$$\Delta z = \frac{1}{g} \int_{r_i}^{r_o} \frac{V^2}{r_c}\, dr$$

$$\Delta z = z_o - z_i = \frac{V^2}{g r_c} (r_o - r_i) \tag{16}$$

where z_i and r_i are the water surface elevation and radius at the inside of the bend, and z_o and r_o are the same parameters at the outside of the bend.

Additional expressions for superelevation can be obtained by assuming that the velocity distribution approximates that of a free vortex, $V = c/r$, or a forced vortex $V = cr$, or combinations of these. However, Equation (16) serves the purpose of illustrating the basic characteristics of flow in bends.

Superelevation in bends produces a transverse velocity distribution which results from an imbalance of radial forces on a fluid particle as it travels around the bend. In Figure 18a, a cross section through a typical bend is shown. The radial forces acting on the shaded control volume are the centrifugal force mv^2/r, and the differential hydrostatic force γdz^2 caused by the superelevation of the water surface dz. As indicated in Figure 18b, the centrifugal force is greater near the surface where the fluid velocity V is greater, and less near the bed where V is small. The differential hydrostatic force is uniform throughout the depth of the control volume. A graphical summation of these two curves (Figure 18b) demonstrates the absence of equilibrium in the fluid in the plane of the stream cross section. This summation indicates the presence of a transverse flow directed toward the concave bank in the upper part of the section and a reverse flow toward the convex bank along the bottom. Transverse currents, superimposed on the longitudinal flow, form the screw-like, helicoidal secondary circulation observed in river bends and laboratory flumes.

Classically, these transverse currents, with a magnitude of about 15 percent of the average current velocity, have been cited as the primary mechanism for scour and deposition in bends. The heuristic argument has been that in the upper portions of the cross section secondary currents increase bankline erosion on the concave bank, and as they flow along the bottom, sweep heavy concentrations of sediment toward the convex bank where deposition forms point bars.

3.2.2.2 Morphology of the Crossing and Pool Sequence. While the existence of transverse secondary currents provides a good first order explanation of the morphology of a bendway, recent research (Hooke, 1974) indicates that their importance has often been overstated. In addition to measuring secondary circulation or helix strength, Hooke studied shear stress and sediment distribution in a laboratory meander bend. Although helicoidal flow influences the distribution of bed shear stress in the bend and is responsible for bringing sediment-free water with high erosive potential to the bed along the concave bank of the bend, Hooke concludes that it is sediment distribution, not secondary currents, which is primarily responsible for the development of a point bar.

Apparently, geometry of the alluvial bed is adjusted to provide, at each point on the bed, precisely the shear stress necessary to transport the sediment load supplied. Hooke observed that the zone of maximum shear stress, and, thus, maximum sediment transport, crosses the tip of the point bar in the upstream part of the bend. This zone traverses the channel centerline in the downstream part of the bend, (Figure 19a) and follows the concave bank to the next point bar downstream. Thus, due to the momentum of the sediment and flowing water, and in spite of secondary flow, most of the sediment in transport crosses the channel centerline and is carried along the concave bank in a zone of maximum shear. The sediment then enters the next bend along its convex bank where it is available for deposition in the low shear zone of the downstream portion of the point bar.

To provide the shear stress necessary to move the sediment, there is a gradual decrease in depth along the concave bank from the pool in one bend to the point bar in the next. This decrease in depth results in a continuous acceleration of the flow and produces high shear stresses. Downstream from the point bar along the convex bank there is a zone of low shear as depth increases and the flow diverges. Decreasing depths in the crossing provide the mechanism for accelerating the flow and producing the shear stress required to move sediment across the bend centerline.

Hooke's conclusion that secondary currents are not the primary mechanism responsible for the existence of point bars is supported by Leopold and Wolman's (1957) observation that no single parcel of water completely crosses the channel in a bend. Friedkin (1945) also observed that most of the sediment scoured from the concave bank of one bend was deposited on the inside of the next bend downstream. Neither observation supports the contention that secondary currents scour sediment from the pool on the outside of a bend and sweep it across the channel to the point bar on the inside.

Figure 19 summarizes the combined effects of secondary currents, sediment distribution, and shear stress distribution, in controlling the morphology of a meander bend. Regions of increasing and decreasing shear stress in the bend based on Hooke's data as well as the primary path of sediment transport through the bend are shown in Figure 19a. As indicated, Figure 19a divides the bend into upstream and downstream sections between A-A', B-B' and C-C'. Using the difference between sediment supply and shear stress as an indicator, the net result in terms of erosion (-), deposition (+), or equilibrium (0) in each region of the bend is summarized in Figure 19b.

Between A and B on the concave bank the bed shear gradually decreases toward the pool, however, secondary circulation balances any tendency toward deposition in this region. Between A' and B' on the convex bank there exists a zone of high sediment supply but also high shear. As a result the upstream section of the bend approaches a state of dynamic equilibrium.

Conditions in the downstream section are quite different. The concave bank from B to C is a region of high shear and low sediment supply accentuated by the erosive power of secondary circulation. Consequently, this is a zone of potentially severe erosion. Along the downstream convex bank from B' to C' high sediment supply and a zone of low shear produce a depositional environment and contribute to point bar growth.

Observations on rivers in many parts of the world have shown that zones of maximum erosion and deposition in a bend do not coincide with the point of maximum curvature (NEDECO, 1959). Various investigators have attempted to quantify this characteristic. Based on observations on the Seine, Fargue estimated that these zones occur one-fourth of the bend length downstream from the point of maximum curvature, and that the crossing also lags the point of inflexion in the transition from bend to bend by about the same length. Lely estimated this lag at about 1.5 times the river width (Leliavsky, 1966).

As a result of the asymmetric distribution of zones of deposition and erosion, a meander loop will tend to migrate laterally across the floodplain and in a downstream direction. The outer bank of the bend recedes through bankline erosion, and the inner bank follows as the point bar deposit grows (Figure 20a), moving the bend laterally across the floodplain. Since zones of both erosion and deposition are skewed toward the downstream section of the bend, a downstream movement is superposed on the lateral migration of the bend. As the process continues, a narrow neck develops and the potential exists for cutoff of the meander loop during a high stage flow. Figure 21 shows schematically the evolution of an idealized sinusoidal pattern into an irregular meandering pattern with remnant meander loops forming oxbow lakes on the floodplain.

3.2.2.3 The Influence of Stage. The influence of river stage on the morphology of the crossing and pool sequence has been recognized by many investigators. In regard to maintaining navigable depths by dredging, Ockerson (1898) observed: "It is not easy to find a satisfactory explanation as to why sediment piles up in ridges instead of being distributed evenly over the bottom. These ridges of sand are usually found on what steamboatmen call crossings; that is, on the path followed by boats when crossing from a pool lying in a bend along one bank to the pool in the bend of the opposite bank. These bars may be piled up to such an extent that during a high or even medium stage their crests may be actually several feet higher than the surface of the water at low stage. The thread of the channel at high stages does not follow the low stage channel but crosses and recrosses it."

Ockerson also noted that as a high stage peaks and begins to fall: "the load is now too heavy for the diminishing velocity and the burden is very rapidly deposited and obstructions are formed which, later, prove serious hindrances to navigation. When the river reaches a low stage, these act as dams to hold the water in pools. The slope on the crossing or dam is thereby increased and likewise the velocity. The crest of the bar consequently cuts out and if this cutting is confined to one channel a good navigable depth may be the result. If the bar is wide and flat there will probably be several insignificant channels, none of which answer the purpose of navigation."

The change in channel cross section between pools and crossings in a meandering river has been sketched in Figure 16. Comparison of the change in flow area at different stages for cross sections in pools and crossings (Figure 22) with the change in water surface profiles

(Figure 16d) provides an explanation of the phenomena observed by Ockerson. At intermediate stages (lines b-b and b'-b' Figure 22) the cross-sectional flow area is approximately the same for both pool and crossing, consequently, the velocity of flow is also nearly the same. During floods, the water level rises to a-a and a'-a', approximately the same height above b-b and b'-b'. Since the width of the crossing is greater than that of the bend, the high stage flow area at the crossing is also greater. With an increased flow area the velocity of flow decreases producing a relatively mild slope and creating a depositional environment on the crossing. The steeper water surface slope over the pool is indicative of a tendency to scour.

At low stage (lines c-c and c'-c'), conditions are reversed. The area at the bend becomes larger than at the crossing, and the crossing bar acts as a natural dam or weir which pools water in the upstream bendway. Increased velocity and increased water surface slope over the crossing tend to produce scour, while the milder water surface slope over the pool contributes to deposition. At high stage, then, the bendway pool scours and deposition occurs on the crossing; however, at low stages the crossing scours and the pool fills.

The changes in cross section and longitudinal profile which accompany changing stage are also reflected in changes in the plan view of the meandering channel, particularly in regard to thalweg location (Figure 16a). While the low-stage thalweg impinges on the concave bank of the bendway, the higher velocities and greater momentum of the high-stage flow tend to "short circuit" the meander pattern. The high-stage thalweg skirts the convex bank and cuts across the tip of the point bar, opening, in some cases, a chute channel across the bar. Thus, changes in stage radically alter the morphology of the crossing and pool sequence by changing both water and sediment flow directions and by modifying patterns of deposition and scour.

Friedkin (1945) investigated the effects of changing stage on source, volume, path of travel, and location of deposition of sand in a laboratory meander bend. Friedkin's results support the previous analysis of the influence of stage on conditions in the natural river. During low stage Friedkin observed that the movement of sand was largely confined to the crossings, where sand was scoured out and deposited in the pools below. As stage increased, bank attack and erosion shifted downstream, and movement of sand began in the bendway. At bankfull stage, the path of the main current was generally across the point bars, and the attack and erosion of concave banks was along the downstream section of the bends. Sand on the upstream parts of the point bars was swept downstream and deposited on the crossings.

Because of the influence of stage on channel morphology, an alluvial river should be viewed as two distinct rivers flowing in a single riverbed. The low-stage river and the high-stage river each has its own character, its own bed forms, its own system of meanders, and its own geometric parameters such as width and depth. This dual personality of alluvial rivers poses serious problems for navigation. The system of channels formed by, and adapted to, the flow conditions of the low-stage river is generally destroyed by the flows of the high-stage river. As stage falls again, the low-flow stream must reform its own channels through a maze of islands, bars, and channels that do not conform to its character and capacity.

Under conditions of falling stage the crossings, which control the least available depth, pose the greatest problems for navigation. This is particularly true when a rising stage is followed by a rapidly falling stage and the water surface drops faster than the sediment deposited on the crossing is removed. This produces a condition of "retarded scour" on falling stages in which there is a time lag between the fall of the water surface and scour of the crossing. In a detailed investigation of navigation conditions on the Niger and Benue Rivers in Nigeria, NEDECO (1959) reported on this phenomena. When the least available depth over a crossing is plotted against water level (Figure 23) a hysteresis curve results which indicates that water levels and bed elevations on crossings are not in phase.

The more direct flow path taken by the high-stage river is not in accordance with characteristics of the low-stage river. The low-stage flow will remold the bed, but with the forces available, change will be slow and gradual. Until the developing low stage channel pattern becomes clear, navigation conditions are uncertain and early decisions regarding location and volume of dredging will be subject to change. In addition, the possibility of subsequent significant change during the same low-water season always exists.

3.2.3 Islands and Side Channels

As pointed out by Rubey (1952), the importance of islands in the morphology of large rivers like the Mississippi has been neglected by most writers. NEDECO (1959) also notes that "little has been published about the origin, shape and life of islands in alluvial rivers in their natural state." In fact, prior to Rubey's analysis of alluvial islands and chutes on the Mississippi and Illinois Rivers, one of the few detailed descriptions of formation and destruction of alluvial islands was presented by Humphreys and Abbot (1876). This was cited in the description of the natural river (Section 3.1.2).

3.2.3.1 Definitions. There are two common types of river islands: rock islands composed of bedrock, and alluvial islands formed by the river with the alluvium that it transports. Only the latter is of consequence to this study. Alluvial islands are distinguished from mid-channel sandbars by vegetative cover which provides a permanence and stability not possessed by the unvegetated bars. Islands divide a reach of river into two or more channels. The larger is referred to as the main or thalweg channel and the smaller channels are side channels.

Side channels which carry appreciable flows at least during high stage, are called chutes. Those that do not carry appreciable flows even at high stage are backwater channels or sloughs. Chute channels and sloughs are further distinguished by bed characteristics. While the bed of a chute channel is composed of about the same materials as the main channel bed, the bed of a slough consists of relatively fine material reflecting the slackwater depositional environment of backwater channels (Simons et al., 1974).

3.2.3.2 Evolution. As previously established, many sections of the Upper Mississippi are more braided in character than meandering. Lane attributed this filling, braided characteristic to processes of aggradation inherited from the Pleistocene. Rubey (1952) attributes the crooked, meandering course of the portion of the Upper Mississippi that he studied not so much to classical meander growth as to division of the main channel by large alluvial islands. The existence of numerous alluvial islands in a rapidly aggrading or sharply degrading stream is not unusual (Rubey, 1952). However, both Lane and Rubey have indicated that the Upper Mississippi is aggrading so very slowly that it can be considered essentially a graded stream. Under the conditions of dynamic equilibrium between sediment load and transport capacity implied by the term "graded," the mechanics of island growth are not obvious.

One striking feature of the morphology of the Upper Mississippi is the fact that islands are larger and more numerous near the mouths of tributary streams. Between Hannibal, Missouri and St. Louis, for example, large islands such as Cuivre, Peruque, and Dardenne have been formed at the mouths of the larger tributaries, the Cuivre River and Peruque and Dardenne Creeks, respectively (Figure 5). The steep gradient and coarse sediments of the Chippewa River produced the delta that formed Lake Pepin. On a smaller scale, continued island growth at the mouths of tributaries between Mosier Landing (RM* 260) and Dardenne Island (RM 227) coupled with the influence of the Missouri River apparently has forced the Mississippi away from the curving trough of the western bluff line and crowded it against the eastern bluff line (Figure 5).

The incidence of numerous large islands adjacent to tributary mouths led Rubey (1952) to postulate a cause and effect relationship. Delta-like sandbars which form at tributary mouths provide the trigger mechanism which initiates island growth. Although most of these bars are transient features, wiped out by the next high flow, occasionally one will build high enough to escape destruction for several seasons. Once vegetation appears, island growth parallels the description of

*River Miles are measured from the mouth of the Ohio River at Cairo, and will be abbreviated RM throughout this study.

Humphreys and Abbot, through stages of willow growth to occupation by dense growths of larger hardwood trees, and finally, formation of a permanent "timber island." Subsequently, larger floods may scour the upstream nose of the island, but this loss is generally compensated by new deposits and willow growth at the lower end. Thus, the island migrates slowly downstream from the point of inception, and another may grow in its place.

A detailed case history of island growth and side channel evolution on the Lower Mississippi was documented by Shull in 1922 and 1924. Shull's description permits delineation of the key elements of the process. During the recession of the 1913 flood, an island formed in an existing chute channel near Belmont, Missouri when a large barge became stranded in the chute. The island continued to grow downstream from the barge in the depositional environment of the chute channel (Simons et al., 1974). Within six years it was three-fourths of a mile long and one-eighth of a mile wide. Its surface area of 60 acres was covered with a growth of cottonwood trees four to eight inches in diameter and 30 to 40 feet tall. Each succeeding flood that inundated the island added to its dimensions. The 1920 flood, for example, deposited 16 to 18 inches of sandy silt over the entire area.

By 1919 the chute between the island and the Missouri bank was beginning to fill. A younger belt of cottonwoods was encroaching from the island side, and willows and small cottonwoods lined the chute along the Missouri bank. Shull revisited the region in 1933 and reported that the chute had closed and the island had become a part of the Missouri mainland. In his words: "The old chute of the river is now occupied by a thick growth of willows..., among which mud deposits have developed to such a depth that the old channel is almost level with the floodplain along the bank of the Missouri shore." The young cottonwoods of 1919 were now 18 inches in diameter and 100 feet tall. Approximately three feet of mud had been deposited on the island in nine years, a rate of four inches per year.

Island growth and side channel destruction required, in this case:

1. The existence of a depositional environment (the original chute channel);
2. A local disturbance to trigger rapid deposition of a sand-bar (the barge);
3. The establishment of vegetation to transform the developing bar into a more permanent island.

Once established, the evolution of the island progressed through stages of:

1. Continued deposition during high stages in the high resistance region created by island vegetation;
2. Encroachment of island and mainland banks and vegetation into the side channel;
3. Deposition in the slackwater of the side channel. As rejuvenation by flood flows became less frequent, the side channel became a slough;
4. Ultimate merger of the island with the mainland floodplain, ending the existence of the side channel.

Prior to 1830 the depositional environment and the trigger mechanism for island growth were often provided by the timber snags which were so hazardous to navigation. The deltaic sandbars at the mouths of tributaries, which Rubey cites as a prime causative factor in island growth, also provide both the necessary depositional environment and trigger mechanism to produce incipient islands.

The topography of many alluvial islands strongly supports the conclusion that most islands are enlarged by aggradation during high stages when sediment laden floodwater encounters the high resistance region created by island vegetation. The process of island growth, once vegetation is established, is similar to that which produces natural levees on the floodplain bordering the river. The velocity of the water flowing through vegetation on the floodplain or island is lower than the velocity which transported the sediment in the river channel. Consequently, much of the sediment carried to the floodplain or island is deposited. The coarsest fractions of the sediment load are deposited near the channel where velocity is first reduced. The finer fractions with much slower settling velocity are carried farther onto the floodplain or island.

On an island which receives high-stage flows from both the main channel and chute channel sides, this formation of a natural levee produces a characteristic topography. The island levee forms a "U" with the base pointing upstream and the arms pointing downstream. Generally, a boggy region of fine-grained deposits can be found in the interior of the island (Yarbrough, 1974). At higher stages the lower, central portion of the island is drowned out and only the peripheral natural levee remains, producing the characteristic "crab claw" shape of many islands in the Upper Mississippi. Perhaps the best example of this topography is provided by Crider Island at RM 280 (Figure 46). The 1880 and 1927 outlines in Figure 46 were made at stages so low that the island interior was not inundated, but the 1973 soundings were made at a high stage, producing the crab claw signature that reflects the processes of island building in the river.

In a 3-by 60-foot laboratory flume Leopold and Wolman (1957) produced an evolutionary sequence of island growth strikingly similar to observations on natural rivers. As described by Leopold, Wolman and Miller (1964) and sketched in Figure 24:

> "In flume experiments conducted in a channel molded in moist but uncemented sand, the introduction into the flowing water of poorly sorted debris at the upper end produced, with time, forms similar in many respects to those observed in the field. After three hours a small deposit of grains somewhat coarser than the average introduced load had accumulated on the bed in the center of the channel. This represented a lag deposit of the coarser fraction which could not be carried [any farther] by the flow..."
>
> "The growth surfaceward of a central bar tends to concentrate flow in the flanking channels, which then scour their bed or erode their banks (or both)...As the cross section is enlarged, the water surface elevation is lowered, and the bar, formerly just covered with water, emerges as an island. In a natural stream the emergent bar may be stabilized by vegetation which prevents the island from being easily eroded and in addition tends to trap fine sediment during high flow. Thus the ground tends to become veneered with silt."

In the natural river, resistant rock or clay outcroppings on one or both sides of the river also contribute to alluvial island growth. The energy shadow downstream of a resistant outcropping along one bank provides the necessary conditions for island formation. Downstream migration induced by erosion at the head and accretion at the tail of the island will result in a series of islands which regularly "drip-off" the projecting part of the riverbank (NEDECO, 1959).

Where bedrock constricts a channel and prevents a section of the river from attaining its average alluvial width, islands usually form in the reach immediately downstream. Islands are formed from material scoured from the contracted section and deposited in the downstream expansion during floods. In the Middle Mississippi, the Thebes Gap reach (Figure 6) is a naturally contracted reach where the width is controlled by rock outcroppings. In Figure 25 Thebes Gap is the reach between grid miles 0 and 3. In 1884 there were six islands in the seven mile reach downstream (grid miles 3 to 10). The thalweg was located west of Burnham Island and east of Powers Island. The numerous side channels in the reach were all classed as chutes. In the intervening years all the side channels except one have disappeared as a result of both natural processes and river contraction works. Powers Island is now joined to the Missouri mainland. The remaining side channel, Santa Fe Chute, although closed by a dike at the inlet and a partial dike at the outlet, has retained its status and has not filled with sediments over the last 90 years. The surface area (Table 2) has remained about the same, since the decrease in width has been compensated for by an increase in length (Simons et al., 1974).

To this point the discussion of island growth and chute channel evolution has not been related specifically to channel pattern. Tributary fans, snags or other obstructions, resistant bankline zones, and bedrock channel contractions can provide the necessary depositional environment and trigger mechanism for island growth in meandering and braided reaches as well as in straight reaches. One additional set of causative factors in the development of a divided reach is related specifically to the meandering channel or meandering thalweg pattern. The short-circuiting of the meander pattern across a point bar during high-stage flows (Figure 16) has been discussed. Once a path has developed across a point bar during high flow, recession of the floodwaters may result in a configuration consisting of a main channel on the concave side of the bend, a middle bar, and a secondary channel on the convex side of the bend (Figure 20b). If the middle bar persists for several seasons and becomes a vegetated island, the channel along the inside of the bend becomes a chute. On a larger scale the complete cutoff of a meander loop (Figure 20a) can also produce the main channel, island, and chute channel configuration.

Simons, Schumm, and Stevens (1974) have documented the history of a divided reach on the Middle Mississippi near Devil's Island (RM 55 to 61). An unusual condition existed in the Devil's Island reach in 1818. The reach was divided into three channels by Devil's Island and Picayune Island, with the main channel on the inside of the bend and Picayune Chute and a smaller chute channel on the outside of the bend (Figure 26a). The configuration of the bend suggests that a point bar cutoff had occurred in the past.

Between 1818 and 1880 the channel upstream of the bend shifted to the west and the main channel on the inside of the bend grew larger. While Picayune Chute remained about the same, the second chute channel decreased greatly in size (Figure 26b). By 1907 a large island had developed in the main channel, and Devil's Island had coalesced with Dusky Bar and Swiftsure Towhead, two smaller islands upstream. Picayune Chute took on the appearance that it has today (Figure 27), and the second chute channel closed at the head and became a slough (Figure 26c). Between 1907 and 1969 the island in the main channel became a part of the Missouri mainland and a number of small islands developed in a dike field along the west bank of the river. Picayune Chute remained essentially unchanged (Figure 26d).

The behavior of the Devil's Island reach was apparently controlled by the westward movement of the upstream reach of the river. Picayune Chute is in a favorable position to receive clear water at its intake which has enabled it to retain its cross-section since 1907. In addition, the large expanse of Devil's Island appears to isolate Picayune Chute from sediment laden high-stage flows of the main channel which might otherwise produce rapid deposition and closure of the chute channel (Simons et al., 1974).

Since the turn of the century literally thousands of rock and pile dikes have been placed in the Mississippi River to contract the flow and improve navigation conditions. As a result, most of the recent side channels formed in the river are man-made. Accordingly, a brief analysis of the role of dike fields in the evolution of islands and side channels is pertinent here. A more detailed examination of the effect of contraction dikes on river morphology and hydraulics is presented in the next chapter.

Simons, Schumm, and Stevens (1974) conducted a detailed study of the relationship between dike fields and side channels both in a laboratory flume and on the Middle Mississippi River. A summary of a field case from the Middle Mississippi illustrates the mechanics of the processes involved. In Figure 28 the various stages of side channel development in a dike field can be seen. The long bar at the bottom of the photograph is a recent development, formed by the extension of older rock dikes. The channel between the bar and the vegetated bank is wide and shallow and has a sand bed. The two older pile dikes that can be seen extending partially into this channel are apparently ineffective.

The backwater channel at the bottom left of Figure 28 is a mature side channel formed by previous dike fields and the island between this old channel and the new channel is well vegetated. There are two trees growing on the dike across the lower end of this backwater channel. Immediately downstream of this dike, the pattern of vegetation indicates that the downstream extension of this side channel has been filled with sediment and covered by vegetation.

The rate of sedimentation between the dikes is very rapid immediately after a dike field is constructed. Local scour at the nose of the dikes supplies the initial sediments. Later, the bar building materials are bed sediments carried along by the river. Once the man-made bar becomes a vegetated island, its evolution is similar to that of a natural island already described. Very little water flows in the man-made side channel between the island and the mainland because the path through the side channel is much more resistant to flow than the main channel. Eventually the side channel becomes a backwater channel as sediments settle in the slackwater environment. Where sediment laden flood flows enter the side channel across the island, the process of deterioration is accelerated. The ultimate fate of the mature side channel is obliteration by both sedimentation and growth of vegetation.

Several conclusions drawn by Simons, Schumm, and Stevens from their analysis of historical changes of islands and side channels on the Middle Mississippi and observed changes in geomorphic models are pertinent to the dredging and disposal problem:

1. Natural backwater channels are a product of the natural, uncontrolled, shifting river. Any river subject to development will experience a deterioration of the natural backwater channels unless these channels are maintained artificially.

2. In the past, the construction of dike fields has eliminated many natural side channels, but these were replaced by side channels resulting from the dike fields.

3. Generally, future channel contraction by extending existing dikes will produce no new side channels.

4. Unless steps are taken to prevent it, ultimately nearly all natural and man-made side channels will fill with sediment and become indistinguishable from the floodplain.

5. Small natural and man-made chute channels fill at a rate of up to three feet a year. Backwater channels fill at rates of between one and five inches a year. Those few large natural chute channels in existence today will remain open for a long period of time.

6. Generally, it is difficult to design dike fields so that the resulting side channels will be self-maintaining. ..ke fields are usually located in depositional areas of the river channel and suitable side channel intakes are not available unless the flow is realigned upstream of the dike field.

7. The life of a side channel can be increased if it can be isolated on the upper end from the main channel. When isolated, the side channel becomes a backwater channel and rates of sedimentation are small.

3.2.3.3 Morphology of Divided Reaches. An understanding of the mechanics of the formation and evolution of islands and side channels is essential to an analysis of the dredging and disposal problem. Knowledge of the morphology and hydraulics of divided reaches is equally important.

Rubey (1952) acknowledged the dilemma posed by the formation of a divided reach in a graded stream. A graded condition in a stream would indicate that no morphologic changes can be permanent that do not somehow contribute to a stream's stability or enable it to maintain a dynamic balance between erosion and deposition. For example, a stream that produces divided reaches because of an increase in sediment load from its tributaries should by this process of division create a more efficient channel to transport the increased load. However, it is not obvious that the smaller channels and less direct course of a divided reach are more efficient than an undivided reach.

The significant change in hydraulic parameters from an undivided to a divided reach has been documented by several investigators. Rubey (1952) analyzed the geometry of six stable island reaches on the lower Illinois River, and Leopold and Wolman (1957) investigated changes on rivers in Wyoming as well as in a laboratory flume. For illustration, results of Rubey's investigations are applied to simplified cross sections in Figure 29 using the average percent change of hydraulic factors observed by Rubey. A number of significant changes are apparent. The total water surface width of the two channels of a divided reach was 16 percent greater than the water surface width of the undivided reach immediately upstream. The average depth in the divided reach was 10 percent less than in the undivided reach, and, consequently, the total relative depth (D/W) decreased markedly. The relative depth of a single branch of the divided reach, however, showed a 55 percent increase. Rubey's data indicates an increase of 37 percent in the wetted perimeter of the divided reach. Similar trends were noted by Leopold and Wolman (1957) on rivers in Wyoming and in laboratory flumes.

The change in depth to width ratio, in particular, suggests a partial explanation of the influence of a divided reach on stream regimen. Gilbert (1914) found that the sediment load a stream can carry varies with the relative depth, that is, if other factors are held constant there is an optimum depth to width ratio for given conditions of discharge and sediment load. Based on Gilbert's observations in a laboratory flume, division of a reach by an island could provide a means of altering the depth to width ratio as an adjustment to varying conditions of sediment load. For a slowly aggrading stream, then, division into several channels would provide a means of optimizing transport capacity and maintaining a graded condition (Rubey, 1952).

The interdependence of the hydraulic and geomorphic variables of alluvial channel flow is nowhere more apparent than in a divided reach. The increase in wetted perimeter (Figure 29) provides a larger frictional surface and, thus, greater resistance to flow in the divided reach. Rubey also indicates a decrease in velocity and an increase in slope through the divided reach (Figure 29). Although increased slope should produce higher flow velocities in the channels adjacent to an island, the combined increase in flow area and resistance apparently account for the lower velocities observed in natural divided reaches. Because of the complexities of flow through an island and chute channel configuration, interference by man, such as closure of a chute channel, may induce an unexpected response and major changes in the morphology of a divided reach.

In general, flows do not divide equally between the main channel and chute channel, nor do water and sediment discharge necessarily divide proportionately. Additional complications are introduced by changes in flow pattern at high and low stage as well as by local conditions such as geometry and alignment of the reach upstream. An excellent example of this complexity is provided by a detailed study carried out by NEDECO (1959) on the Long Island reach of the Niger River (Figure 30). The islands in the reach can be attributed to the presence of a region of resistant clay on the right bank (Point A) which forms a resistant outcropping and creates the necessary conditions for island formation. In this study sediment discharge (bed load) and water discharge measurements were taken at both low and bankfull stage, and provide a unique record of the variation of flow conditions with stage in a divided alluvial reach.

At a low-stage discharge of 190,000 cfs (Figure 30a) a slightly larger water discharge (54 percent) passes through the thalweg channel to the left of Long Island. The proportionality of sediment flow, however, is just the reverse with almost 60 percent transported through the chute channel. This division of the sediment load coincides with Hooke's analysis of shear and sediment distribution in a bend which places the heavy sediment concentrations along the inside of the bend.

Bankfull stage alters conditions in the divided reach significantly (Figure 30b). At 470,000 cfs the water flow around Long Island is split almost evenly between the main channel on the left (52 percent) and the chute channel on the right (48 percent), showing only a slight tendency for the higher flow to short circuit the bend of the meandering thalweg. The bulk of the sediment load (66 percent), however, passes through the main channel to the left of Long Island at high stage. In this particular case the division of sediment load is certainly influenced by the large sediment transport rates of the chute channels along the left bank between the Walker Islands upstream (Figure 30).

Application of the continuity principle to sand transport through the Long Island reach supports earlier conclusions relative to the effect of stage on the crossing and pool sequence. At low stage almost 14,000 ft^3/day of sand are scoured from the crossing between mile 245 and mile 246, while at high stage, almost 275,000 ft^3/day deposit in the same section. At low flow the sand scoured from the crossing above the divided reach is carried along the two channels on either side of Long Island and is dropped again near mile 244 in the right channel and at the downstream end of the left channel. This low-stage deposition downstream of the divided reach could threaten the integrity of a navigation channel. Much of the sand that remains in the mile 245-246 section at bankfull stage deposits at mile 245 in front of the entrance of the Long Island right channel, blocking it. At flood stage (600,000 cfs) this material is carried into the right channel, and with floods of sufficient duration, is transported through the channel, opening it again. Thus, the right channel acts as a chute for the transport of water and sediment during flood stages. The significant imbalance of scour and deposition at low and high stage upstream of the divided reach would create serious problems relative to the maintenance of a navigation channel.

The key role of the geometry and alignment of the reach immediately upstream can be seen in the case of a divided reach created by cutoff of a point bar in a bend. In Figure 31 the average slope of the chute channel across the point bar may be twice that of the bend-way channel. This increase in slope together with the favorable angular orientation of the chute channel results in a large percentage of the flow and large quantities of sediment being diverted through the chute at high flow, producing a wide, shallow channel. As stages recede the depth in the chute channel decreases and resistance to flow is such that the deep, narrow bendway channel is more efficient for the transport of water and sediment. The greater percentage of water and sediment flow then returns to the bendway channel (Haas, 1963).

3.2.4 Sediment Transport

The sediment in a river has its origin in the drainage basin. Eroded material is carried into the river and along the river's course by flowing water. The ultimate fate of this material is deposition in the lower reaches of the river, on the river delta, or for the finer material, in the sea. This constant displacement of material implies a slow but continuous change in the longitudinal profile of the river, ending eventually in the destruction of the upland region drained by the river. As a result, it must be anticipated that large quantities of sediment will pass through a river system each year.

In regard to the transport of sediment by a stream Einstein (1950) observed:

> "Every sediment particle which passes a particular cross section of the stream must satisfy the following two conditions: (1) It must have been eroded somewhere in the watershed above the cross section: (2) it must be transported by the flow from the place of erosion to the cross section. Each of these two conditions may limit the sediment rate at the cross section, depending on the relative magnitude of two controls: the availability of the material in the watershed and the transporting ability of the stream."

The quantity of sediment brought down from the watershed depends on the geology and topography of the watershed; magnitude, intensity, duration, and distribution of rainfall; vegetative cover; and the extent of cultivation and grazing. These variables are subject to so much fluctuation that the quantitative analysis of any particular case is extremely difficult. It is possible, however, to use regression methods to develop a soil loss relationship for a given area from long term sediment discharge records.

In regard to the second condition cited by Einstein, the capacity of a stream to transport sediment depends on hydraulic properties of the

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stream channel. Such variables as slope, roughness, channel geometry, discharge, velocity, turbulence, fluid properties, and size and gradation of the sediment are closely related to the hydraulic variables controlling the capacity of the stream to carry water, and are subject to mathematical analysis.

The transport of sediment has been alluded to in general terms in previous sections. The purpose here is to provide more detail relative to those definitions and concepts of sediment transport required in the analysis of dredging and disposal problems.

3.2.4.1 Definitions. Sediment in transport in a stream can be classified by several criteria including: sediment source, mode of transport, and data collection limitations.

1. Sediment classification by source of the sediment:
   - (a) Bed-material load: that part of the total sediment discharge which is composed of grain sizes represented in the bed--equal to the transport capacity of the flow.
   - (b) Wash load (fine material load): that part of the total sediment discharge which is composed of particle sizes finer than those represented in the bed--determined by available bank and drainage area supply rate.
2. Sediment classification by mode of transport:
   - (a) Bed load (Contact load): that part of the total sediment discharge that moves by rolling or sliding along the bed.
   - (b) Suspended load: that part of the total sediment discharge that is supported by the upward components of turbulence and that stays in suspension for an appreciable length of time.

3. Sediment classification by data collection limitations:

Measured and unmeasured load: due to the design of the various depth integrating sediment samplers, there is a physical constraint on the depth to which a sample can be taken. Most sediment samplers can measure to within .3 ft of the bed. That portion of the cross section above this point is termed the measured zone and below the unmeasured zone.

The total sediment load of a stream is the sum of the bed-material load and the wash load, or bed load and suspended load, or measured and unmeasured load.

Sediment particles are generally classified by size and can be grouped in the following ranges (Rouse, 1950):

64 mm and larger	-	cobbles to boulders
2 mm to 64 mm	-	gravel
.062 mm to 2 mm	-	sand
.004 mm to .062	-	silt
.00024 mm to .004 mm	-	clay

Figure 32 shows the vertical distribution of suspended sediment for different particle size groups from one sediment sample.

The vertical distribution of suspended sediment can be described by the equation:

$$\frac{C}{C_a} = \left[\frac{D-y}{y} \frac{a}{D-a} \right]^z \qquad (17)$$

where

C = the concentration at a distance y from the bed

C_a = the concentration at a point a above the bed

D = the depth of flow

$z = \omega/\beta\kappa v_*$ (the Rouse number)

ω = particle fall velocity

β = a coefficient relating diffusion coefficients

κ = the von Karman velocity coefficient

v_* = the shear velocity ($\sqrt{gRS}$)

Figure 33 shows a family of curves obtained by plotting equation 17 for different values of the Rouse number z. It is apparent that for small values of z, the sediment distribution is nearly uniform. For large z values, little sediment is found at the water surface. The value of z is small for large shear velocities v_* or small fall velocities ω, and large for small v_* and large ω. Thus for small particles or for extremely turbulent flows, the concentration profiles are nearly uniform.

For fine particles the value of β is approximately equal to one. The value of κ is often taken as 0.4, though κ decreases with increasing sediment concentration. Sediment concentration is conveniently expressed as concentration by weight:

$$C = \left[\frac{\text{weight of sediment}}{\text{weight of water-sediment mixture}} \right] \tag{18}$$

The units commonly used to express concentration are parts per million (ppm):

$$C \text{ in ppm} = (C \text{ by weight})(1{,}000{,}000) \tag{19}$$

The particle fall velocity, ω, used in equation 17 is the primary indicator of the interaction between bed material and the fluid. The fall velocity of a sediment particle is defined as the velocity of that particle falling alone in quiescent, distilled water of infinite extent. The physical size of the bed material, as measured by the fall diameter or by the sieve diameter, is an important factor in determining fall velocity. Use of the fall diameter (defined as the diameter of a sphere with specific gravity of 2.65 that would have the same fall velocity as the sediment particle) instead of the sieve diameter is advantageous since the shape factor and density of the particle can be eliminated as variables. That is, if only the fall diameter is known, the fall velocity at any temperature can be computed; whereas the same computation when the sieve diameter is known requires knowledge of the shape factor and density of the particle.

3.2.4.2 Bed-Material Transport. As implied by the definitions, the distinction between bed-material load and wash load is of importance to the engineer. Bed material is transported at the capacity of the stream and is functionally related to measurable hydraulic variables. Wash load is not transported at the capacity of the stream, depending instead on availability, and is not functionally related to hydraulic variables. While there is no sharp demarcation between wash load and bed-material load, one rule of thumb assumes that the bed-material load consists of sizes equal to or greater than .062 mm, the division between sand and silt. Another reasonable criteria is to choose a sediment size finer than the smallest 10 percent of the bed material as the point of division between wash load and bed-material load.

Sediment particles which constitute the bed-material load are transported either by rolling or sliding along the bed (bed load or contact load) or in suspension. Again there is no sharp distinction between contact load and suspended load. A particle of the bed-material load can move part of the time in contact with the bed and at other times be suspended by the flow. Generally, the amount of bed material moving in contact with the bed of a large sand bed river is only a small percentage of the bed material moving in suspension. These two modes of transport follow different physical laws which must be incorporated into any equation for estimating the bed-material discharge of a river.

The fine material moving as wash load usually will not pose direct problems to dredging operations in the riverine environment. This is not true, however, of dredging in the estuarine or marine environments. At large concentrations the fine material can influence the capacity of a stream to transport bed material through its influence on fluid properties such as viscosity and density. Schumm (1960) found that the shape of many stable channels is closely related to the percentage of silt and clay (fine material) in the sediments forming the perimeter of the channel. The width to depth ratio (F) is related to the percentage of silt-clay (M) by the equation:

$$F = 255\ M^{-1.08} \tag{20}$$

where:

$$M = \frac{Sc \times W + Sb \times 2D}{W + 2D} \tag{21}$$

Sc = silt clay percentage in the bed

Sb = silt clay percentage in the banks

W = width

D = depth

Thus, M is the weighted mean percentage of fine sediments in the material composing bed and banks. To the extent that channel shape influences dredging, and vice versa, wash load can indirectly impact the dredging problem.

That portion of the bed material that moves through a river system in suspension is maintained in suspension by the turbulent fluctuations of the flow. Again, this suspended material should not influence dredging activities directly so long as it remains in suspension. In regions of reduced velocity and turbulence, however, particle fall velocity will tend to exceed the upward components of turbulence and the quantity of bed material in suspension will decrease while contact load increases. It is as bed (contact) load that bed material exercises its greatest influence on river form, character and resistance, and consequently, most directly influences dredging operations.

Bed-load motion has been the subject of many analytical as well as experimental studies. Since bed load moves predominantly in the unmeasured zone, the determination of quantities actually transported by rivers is difficult. The principal difficulty encountered in the measurement or estimation of bed-load transport is the discontinuous nature of bed-load motion. Sand grains in contact with the bed move suddenly and erratically by rolling or sliding, with periods of motion being followed by periods of rest. As a result, statistical analysis of a large number of separate measurements is required to obtain a reasonable estimate of bed load quantities in a river.

Lane and Borland (1951) presented a method of estimating the amount of bed load as a percentage of the suspended sediment load. Since the rationale supporting this method provides an excellent summary of the factors influencing bed-load movement, it is sketched briefly here. Although very few quantitative measurements of the total load have been made to serve as a guide in estimating bed load, application of a few general concepts to the problem will limit the error in making such an estimate. The three major variables which affect the amount of bed load a stream can carry are: (1) the size of the bed material (the fall velocity), (2) the slope of the stream or average stream velocity, and (3) the nature of the channel (depth, size, shape and roughness of bed and banks).

To aid in the evaluation of the effect of these variables on bed load Lane and Borland provide the following criteria:

1. Smaller concentrations of suspended material usually imply higher percentages of bed load.
2. The ratio of bed load to suspended load is usually larger for low or medium stages than for high stage. Thus, a stream in which flow does not fluctuate widely is likely to carry a larger percentage of bed load.
3. Streams with wide shallow channels carry a higher proportion of sediment as bed load than streams with deep narrow channels.
4. Streams with a high degree of turbulence tend to have smaller amounts of bed load.
5. The nature of the source of sediments influences the magnitude of the bed-load correction, that is, the occurrence of large quantities of coarse material on the watershed, so located that it can be moved easily into the channel is indicative of higher percentages of bed load.

From this discussion it is apparent that the number of variables involved in the estimation of bed load make it difficult to devise a simple relation which includes them all. Lane and Borland recommend a table suggested by Maddock (Table 3) as an aid in estimating bed-load percentages. Although this table does not incorporate all the pertinent variables, it does provide an estimate which can be tempered by a knowledge of the other parameters affecting bed-load transport.

The ratio of bed load to total load in a stream was used by Schumm (Shen, 1971b) as a basis for classifying alluvial channels as suspended-load, mixed-load, or bed-load channels. Table 4 summarizes Schumm's classification scheme which also includes both channel stability and the percent silt-clay, as defined in equations (20) and (21), as parameters. This approach to channel classification implies that the type of material transported or its mode is a major factor determining the character of the stream channel. Absolute size of the sediment load is apparently less important than the manner in which it moves through the channel.

There have been many equations developed for the calculation of bed-material transport. The variation in the quantity of bed-material transport predicted by these equations is significant, with different methods yielding results that differ by a factor of as much as 100. Given the number of variables involved, their interdependence, and the statistical nature of bed-material transport, this difference should not be surprising. If the various methods are applied with a knowledge of the hydraulics and morphology of the river in question, and if the limitations of the method selected are recognized, useful results can still be obtained.

Because bed material is transported as both suspended load and contact load the different physical laws of these modes of transport must be incorporated into any method for predicting total transport of bed material. Transport of bed or contact load is usually related to the tractive force or shear on the bed as in the classic Du Boys formula:

$$q_s = C_s \tau_o (\tau_o - \tau_c) \qquad (22)$$

where

q_s = sediment transport per unit width of section per unit time

C_s = a sediment parameter

τ_o = intensity of bed shear

τ_c = critical shear at which motion is initiated

The Shields formula is a well known relation of the Du Boys type:

$$q_s = 10\, q\, S \frac{(\tau_o - \tau_c)}{\gamma(S_s - 1)d} \qquad (23)$$

where

q = water discharge per unit width

S = slope of the energy gradient

S_s = specific gravity of the sand

γ = specific weight of the fluid

d = diameter of the bed material

Meyer-Peter and Muller (Sheppard, 1960) developed a bed-load equation based on experiments with sand particles of both uniform and mixed size, natural gravel, lignite, and baryta. Their equation as modified by the United States Bureau of Reclamation for units generally in use in the United States and assuming transport of quartz particles in water takes the form:

$$q_b = 1.606\,[3.306\,(\frac{Q'_b}{Q})(\frac{d_{90}^{1/6}}{n_b})^{3/2}(DS) - 0.627\, d_m]^{3/2} \qquad (24)$$

where

q_b = contact load transport in tons per day per foot of width

Q'_b = water discharge quantity determining bed-load transport

Q = total water discharge

d_m = effective diameter of the sediment in mm

n_b = a roughness coefficient

D = depth of flow

S = energy slope

The Meyer-Peter and Muller equations can be recast into the form

$$q_b = \kappa(\tau - \tau_c)^{3/2} \tag{25}$$

In this form its similarity with the Du Boys or Shields tractive force relationship is apparent. The Meyer-Peter and Muller equation is applicable to streams with little or no suspended-sediment discharge and is widely used for gravel and cobble bed streams.

The suspended bed-material discharge for steady, uniform, two-dimensional flow is:

$$q_s = \gamma \int_a^D v \; c \; dy \tag{26}$$

where

v = time averaged flow velocity at distance y above the bed

c = time averaged sediment concentration at distance y above the bed

a = a distance above the bed--usually taken as the thickness of the bed layer (see Figure 33)

D = depth of flow

Typical velocity and concentration profiles are shown in Figure 34.

To integrate equation 26, v and c must be expressed as functions of y. The one-dimensional diffusion equation (equation 17) can be used to describe the concentration profile, and a logarithmic velocity distribution is generally used for the velocity profile:

$$\frac{v}{v_*} = 2.5 \ln 30.2 \frac{xy}{k_s} \tag{27}$$

where:

v = local mean velocity at depth y

v_* = shear velocity

x = Einstein's multiplication factor

k_s = height of the roughness elements on the bed (d_{65} of bed material for sand bed channel)

Substitution of the velocity and concentration profiles into equation (26) yields:

$$q_s = \gamma \, v_* \, C_a \int_a^D \left[\frac{a}{D-a} \cdot \frac{D-y}{y}\right]^z \left[2.5 \ln 30.2 \frac{xy}{k_s}\right] dy \tag{28}$$

This equation has been integrated by many investigators to provide an estimate of that portion of the bed-material load which moves in suspension.

Einstein's (1950) method for calculating total bed material transport, the well known bed-load function, sums up the contact load and the suspended load. Einstein assumed that the probability of any particle of the contact load moving in a given unit of time could be expressed in terms of the rate of transport, size and relative weight of the particles, and a time factor equal to the ratio of the particle diameter to its fall velocity. The same probability was expressed in terms of the ratio of the forces exerted by the flow to the resistance of the particle to movement. The two forms were equated in a general function of the form $\phi_* = f(\psi_*)$, (Bondurant, 1970).

For the suspended load Einstein integrated his form of equation (28). He then established a relationship between the suspended sediment load and the contact load by proving that there is a constant exchange of particles between the two modes of transport. Einstein's bed-material discharge function gives the rate at which flow of any magnitude in a given channel will transport the individual sizes which constitute the bed material. For each size, d, of the bed material the contact load is given as $i_b q_b$, and the suspended load is $i_s q_s$. Thus the total bed-material discharge is

$$i_t q_t = i_s q_s + i_b q_b \tag{29}$$

where i_t, i_s, and i_b are the fraction of the total, suspended, and contact loads, q_t, q_s, and q_b, for a given grain size d. Using the continuous exchange of particles to relate suspended and contact loads, equation (29) becomes:

$$i_t q_t = i_b q_b (1 + P_E I_1 + I_2) \tag{30}$$

where P_E is a transport parameter and I_1 and I_2 represent Einstein's integration of the suspended load relationship. Equation (30) gives the capacity of a stream to transport bed-material load under steady, uniform flow conditions. Einstein's bed-load function represents the most detailed and comprehensive treatment, from the point of fluid mechanics, that is presently available; however, its application is somewhat complicated (Graf, 1971).

Colby (1964) has proposed a simple, effective method for computing bed-material discharge. Guided by Einstein's bed-load function and relying heavily on data from natural streams and laboratory flumes, Colby developed graphical relations among depth of flow, mean velocity, and bed-material discharge (Figure 35). With an uncorrected sediment discharge, q_n, obtained from depth, velocity, and median size of the bed material (d_{50}) and Figure 35, the total discharge is obtained from:

$$q_t = [1 + (k_1 k_2 - 1)(.01)k_3]\ q_n \tag{31}$$

where k_1, k_2, and k_3 correct for temperature, concentration of fine sediment, and size of bed material, respectively.

In his 1950 publication Einstein presented an illustrative example which applied the bed-load function to a test reach of Big Sand Creek, near Greenwood, Mississippi. The Colby method has also been applied to this same reach and the sediment rating curves resulting from these calculations are compared in Figure 36. Reasonable agreement between the results of the two methods is obtained over the range of calculations. The variation in the results is partially explained by the fact that in the test reach selected most of the bed material moves in suspension.

Chien (1954) has shown that the Meyer-Peter and Muller equation (24) can be written in the form:

$$\phi_* = \left(\frac{4}{\psi_*} - 0.188\right)^{3/2} \tag{32}$$

Equation (32) is compared with Einstein's $\phi_* = f(\psi_*)$ relationship for computing contact load in Figure 37. The agreement is excellent.

Figure 36 points out the strong dependence of sediment transport on discharge and velocity. The sediment discharge increases rapidly with increasing water discharge. Virtually all methods for estimating the movement of bed material agree on the basic point that sediment transport increases more than proportionally with increasing velocity. For example, Gilbert's (1914) work indicates that for constant slope, transport capacity varies on the average with the 3.2 power of velocity.

3.2.4.3 Bed Forms and Resistance. The bed of an alluvial river seldom forms a smooth regular boundary, but is characterized instead by shifting forms that vary in size, shape, and location under the influence of changes in flow, temperature, sediment load, and other variables. These bed forms constitute a major part of the resistance to flow exhibited by an alluvial channel, and exert a significant influence on flow parameters such as depth, velocity, and sediment transport. While the detailed mechanics of the interrelations involved are essentially unknown, it is recognized that variation in bed forms permits an internal adjustment of a channel to accomodate relatively large changes in discharge, sediment load, and other variables without requiring a corresponding change in other channel boundary conditions (Bordurant, 1970).

The bed forms that may occur in an alluvial channel are plane bed without sediment movement, ripples, ripples on dunes, dunes, plane bed with sediment movement, antidunes, and chutes and pools. These bed configurations are listed as they occur sequentially with increasing values of stream power (τV or γDSV) for bed material with a d_{50} less than 0.6 mm. For coarser bed material, dunes form instead of ripples after the beginning of motion. Typical bed forms and their relation to the water surface (in phase or out of phase) are shown in Figure 38. These bed forms are not mutually exclusive occurrences in time and space in a stream. They may form side by side in a cross section or reach producing a multiple roughness, or they may occur in sequence producing a variable roughness (Simons and Richardson, 1971).

Using these bed forms as a basic criteria, flow in alluvial channels is divided into two regimes of flow separated by a transition zone. These two flow regimes are characterized by similarities in the shape of the bed form, mode of sediment transport, process of energy dissipation, and phase relation between the bed and water surface. These two regimes and their associated bed forms are:

1. Lower Flow Regime--small stream power
    a. Ripples
    b. Ripples superposed on dunes
    c. Dunes
2. Transition Zone
3. Upper Flow Regime--large stream power
    a. Plane bed (with sediment movement)
    b. Antidunes
    c. Chutes and Pools

In the Lower Flow Regime resistance to flow is high and sediment transport is small. The water surface undulations are out of phase with the ripples or dunes which constitute the bed, and there is a relatively large separation zone downstream from the crest of each ripple or dune. The resistance to flow is primarily form roughness. The most common mode of bed-material transport is movement of individual grains up the back of a ripple or dune then avalanching down its face.

After coming to rest on the downstream face of the ripple or dune, the particles are buried and remain at rest until exposed again by the downstream movement of the dune.

In the Upper Flow Regime resistance to flow is low and sediment transport is large. The most common bed forms are plane bed or antidunes and the water surface is in phase with the bed surface except when an antidune breaks. Normally, there is little separation of the fluid from the bed surface. Resistance to flow is primarily the result of grain roughness; however, wave formation and subsidence and energy dissipation when antidunes break also contribute to resistance. The dominant mode of sediment transport is continuous rolling of individual grains downstream in sheets several grain diameters thick. Antidunes are so named because under certain conditions they can move upstream against the flow. They form as trains of waves that gradually build up from a plane bed and plane water surface, and may break like surf as they become unstable (Figure 38). As antidunes break large quantities of bed material can be briefly suspended, stopping momentarily the continuous motion of sediment particles associated with upper regime flow.

In the Transition Zone the bed configuration is erratic, ranging between conditions of lower and upper regime flow, as dictated primarily by antecedent conditions. Resistance to flow and sediment transport also exhibit the same variability as bed form in the transition zone. In many instances of transition flow the bed configuration oscillates between dunes and plane bed.

Simons and Richardson (1966) developed a graphical relation among stream power (τV), median fall diameter, and bed form using both flume and stream data. This relation (Figure 39) can give an indication of the form of bed roughness to be anticipated if the depth, slope, velocity, and fall diameter of the bed material are known. Another useful graphical relation (Figure 40a) shows schematically the effect of bed form on a roughness coefficient such as Manning's "n". As the bed configuration sequences through lower regime to upper regime, Manning's "n" changes from a typical value of 0.012 to 0.014 for plane bed without sediment motion, to values as high as .04 for a dune bed. Increasing stream power and transition to upper regime, plane bed conditions can produce a decrease in roughness to values as low as .010 to .015. The consequent effect on flow velocity can be seen in Figure 40b.

In a natural stream it is possible to experience a large increase in discharge with little or no change in stage as a result of a shift in bed configuration from dunes to plane bed. Figure 41 shows a typical break in the depth-discharge relation resulting from this phenomena. Conversely, several investigators have shown that an increase in depth, with constant slope and bed material, can change a dune bed to plane bed or antidunes, and that a decrease in depth can reverse the process (Simons and Richardson, 1971).

3.2.5 Summary

The complexity of alluvial channel flow and the dynamic nature of river systems are reflected in the large number of interrelated variables necessary to describe flow in natural streams. The evolution of channel pattern and geometry together with continuous changes in bed form in response to changing conditions of water and sediment discharge also add to the complexity of river systems.

A river like the Mississippi above Cairo consists of a continuum of channel patterns with straight, braided, and meandering reaches intergrading along its length. Each of these channel configurations poses different problems for the river engineer interested in maintaining a navigable waterway, and an understanding of their general characteristics is essential to an analysis of dredging and disposal problems. Equally important, qualitative relationships such as Lane's (Equation 11) provide a valuable tool to the river engineer. Where quantitative data is available for detailed analysis of long term response to engineering works, qualitative analysis constitutes an essential first step in the analysis by establishing the general trends to be anticipated. As is more often the case, when time or data limitations preclude a detailed quantitative analysis, the indicators provided by qualitative relationships are even more valuable.

Certain aspects of the morphologic scene are of central importance to the dredging problem, including the crossing and pool sequence, the island and side channel configuration, and the transport of bed-material load. Examination of the hydraulics of flow in bends reveals the existence of a secondary, helicoidal circulation that influences bendway morphology. Recent research indicates that the dominant factors that control the morphology of the crossing and pool sequence in a bend are the distribution of shear stress and sediment concentration. Variations in stage exert a significant influence on flow direction and sediment source and deposition regions in an alluvial bendway.

The difference in character, meander system, and geometric parameters between the low-stage and high-stage river lead to the conclusion that an alluvial river is really two rivers flowing in the same bed. This dual personality of an alluvial river poses serious problems for the design and maintenance of engineering works which, although fixed in form and location, must accomplish their intended purpose under conditions of continuous change. Because of the transitory nature of both dredged cuts and unconfined disposal sites, dredging operations to maintain a navigable channel are certainly as seriously impacted by this variability as any engineering works on a river. The radical change in bed form, resistance to flow, and mode of sediment transport that accompanies the transition from lower regime to upper regime flow underscores the dual character of an alluvial river.

A portion of this study is directed at the impact of dredged material disposal on sloughs, chute channels, and backwater areas, accordingly, a detailed consideration of the evolution and morphology of islands and side channels contributes to the necessary baseline for analysis. In general, island growth requires the existence of a depositional environment coupled with a local disturbance to trigger deposition. The conditions at tributary confluences and downstream of resistant areas along one or both banks satisfy these criteria, and frequently result in formation of incipient islands. Prior to men's intervention, the numerous timber snags found on the river often triggered island growth, and more recently, construction of numerous dikes to contract the river has provided both the depositional environment and the trigger mechanism that produce alluvial islands. Another phenomena, point bar cutoff in a meander loop, also forms an incipient island. Once vegetation is established on an emergent bar, island growth proceeds as coarse sediments are deposited in natural levees on the periphery of the island, and finer material is washed toward the interior. Pioneer vegetation such as grass and willow is succeeded by hardwood species such as oak and maple, and the island becomes a "permanent" feature of the river channel.

The side channel formed by island growth also experiences an evolution, but one of gradual decay. The open chute channel gradually deteriorates as vegetation encroaches from both island and mainland, and sediments fill the channel. The orientation of the side channel and the configuration of the upstream reach are key elements in determining the life of a side channel, but the ultimate fate of the mature side channel is obliteration. Significantly, the life of a side channel can be prolonged by isolating it at the upper end from the main channel. The rates of sedimentation in the closed backwater channel are much slower than in the open chute channel.

An analysis of the hydraulic parameters of divided reaches suggests that they play a significant role in stream morphology. The increase in the depth to width ratio of a single branch of a divided reach, for example, is apparently an adjustment toward a more efficient cross section for the transport of sediment. Division of a reach into several channels by island formation would then represent one process by which an alluvial channel approaches the graded condition. The unequal and disproportionate division of water and sediment flow between main channel and side channel is an indicator of the complexity of flow in a divided reach. The fact that water and sediment distribution in a divided reach changes with rising and falling stage contributes to this complexity. Any proposal to modify the configuration of a divided reach by closure of a chute channel with dikes or dredged material, for example, must be thoroughly analyzed. Without such analysis there is no assurance that the response induced will be the one desired.

The transport of sediment through the system is the most critical aspect of any river development plan. This is true regardless of the intended purpose of development: flood control, power generation, irrigation, or navigation. Any development scheme that does not assure that the system is capable of transporting the requisite volumes of both water and sediment will face monumental problems. While the wash load in the system does not normally pose a direct problem to dredging operations or other development work, its influence on fluid properties and channel geometry does impact the capacity of a channel to transport bed material. Although 80 to 90 percent of the bed-material load normally moves in suspension, this suspended material does not play the dominant role in establishing river morphology. It is as bed (contact) load that bed material exercises its greatest influence on river form, bed characteristics, and resistance to flow, and consequently, most directly influences dredging operations.

The bed or contact load can be estimated as a portion of the suspended load or calculated by methods which are generally based on tractive force concepts. The Meyer-Peter and Muller equation and the contact load portion of the Einstein bed-load function are the best known and most widely used of these methods. Any method for estimating total bed-material transport must recognize the different physical laws that govern transport in suspension and movement in contact with the bed. Einstein's (1950) bed-load function which combines probability concepts with a detailed analysis of the mechanics of the problem offers the most comprehensive treatment. However, Colby's (1964) graphical approach provides results which are sufficiently accurate for many engineering situations.

The complexity of alluvial channel flow is nowhere more evident than in the influence of bed form on resistance to flow, mode of sediment transport, and on hydraulic parameters such as velocity and depth of flow. The widely differing characteristics of lower and upper regime flow must be considered in the analysis of river engineering problems.

Chapter 4

THE EFFECTS OF DEVELOPMENT

Prior to 1930 development of the Mississippi River between Cairo and St. Paul for navigation followed the central theme of contraction, revetment, and dredging where necessary to attain the desired depth. After 1930 the development of the Mississippi between St. Louis and St. Paul followed the radically different scheme of utilizing low dams and locks, supplemented by dredging, to attain navigable depths. Because of this difference, the analysis of the geomorphic and hydraulic response of the river to man's activity on the Upper Mississippi (St. Paul to St. Louis) is developed separately from the analysis of the response on the Middle Mississippi (St. Louis to Cairo).

The great length of the Upper Mississippi, 645 river miles from St. Louis to St. Paul, precludes a detailed analysis of the entire river; however, a rational basis for selecting a detailed study area is available. The Mississippi River basin above Cairo was divided into 20 land resource areas (LRA) to facilitate the various analyses of the Upper Mississippi River Basin Coordinating Committee (1972). The dominant physical characteristics that define an LRA include: land use, elevation and topography, climate, water, and soil type. The sediment yields from these LRA provide a basis for selecting a limited segment of the Upper Mississippi as a detailed study area. Sediment yields are compared graphically in Figure 42 and indicate that the yields in the extreme southern portion of the basin are 200 to 250 times greater than those in the northern portion of the basin. Land Resource Area 115 which includes the Mississippi from RM 0 at Cairo to RM 375 above Keokuk, Iowa has a sediment yield of almost 2500 tons per year per square mile for a representative drainage area of 35 square miles, almost 2.5 times that of the adjacent upstream LRA (Figure 42).

On the assumption that sediment transport will have the greatest impact on engineering activities, particularly dredging, in regions of high sediment yield, a detailed study area for the Upper Mississippi is most logically located in LRA 15. Consequently, the analysis of response to development on the Upper Mississippi is concentrated in the reach between Lock and Dam 26 at Alton, Illinois (RM 202.9) and Lock and Dam 22 (RM 301.2), eight river miles south of Hannibal, Missouri. This 98 mile reach of river, hereafter referred to as the detailed study area, encompasses Pools 24, 25, and 26 of the Upper Mississippi lock and dam system (Figure 5). It should be noted that trends established in the southern portion of the Upper Mississippi basin may not be completely representative of processes at work in the lower sediment yield environment of the northern portions of the basin.

On the Middle Mississippi (Figure 6) with only 195 miles of river, conditions are less complicated. Response in the vicinity of St. Louis, with its long term gaging station, and in a 14 mile prototype reach from RM 140 to RM 154 which has been studied in detail by the Corps of Engineers, are assumed to be representative of response to man's activities between St. Louis and Cairo.

4.1 Development of the River

4.1.1 Early Development

Man's activities on the Mississippi were mentioned briefly in Chapter 3 in connection with the definition of the natural, pre-1900, river. In the 75 years of development since the turn of the century the Mississippi has been, according to Rhodes, (1972): "Damned, leveed, jettied and polluted 'till huck finn himself wouldn't recognize it." Limited efforts to improve navigation conditions on the river began considerably earlier. In 1824 Congress authorized the removal of snags and other local obstructions such as shoals, sandbars. and rock in several reaches of rapids. Shreve had essentially eliminated the snag problem by the early 1830's (Section 3.1.2). Between 1836 and 1840 Lieutenant R.E. Lee, Corps of Engineers, designed the first channel stabilization works on the Middle Mississippi. Two dikes were constructed in the vicinity of St. Louis to direct the current of the river so as to remove a large sandbar in front of the harbor.

The River and Harbor Act of 1878 constituted the first comprehensive plan to improve navigation on the upper river and authorized a 4½-foot channel from the mouth of the Missouri to St. Paul. The 4½-foot channel was to be achieved by closure of chutes, bank revetment, and contraction of the channel by wing dams (dikes). Prior to initiating construction, a comprehensive hydrographic survey was made during 1878-79. This survey provided the base for the 1891 hydrographic charts used in this study.

In 1881 a comprehensive project plan for continuous channel improvement on the Middle Mississippi was initiated by the Corps of Engineers. Work progressed downstream from St. Louis, using bankline revetment and permeable dikes to reduce the river to a width of 2500 feet and obtain a 200 foot wide, 8 foot deep navigation channel. The project also included measures to reduce or eliminate flow through sloughs and chute channels to confine the river's flow to the main channel.

It was in this same period that organized levee construction for flood protection was begun on the Middle Mississippi. The 1879 Illinois State Drainage and Levee Act cleared the way for organized levee districts to accomplish the needed works with the aid of state funds. Levee construction on the Middle Mississippi was not intensive, however, until 1907. By 1973 almost the entire Middle Mississippi between the mouth of the Missouri and Thebes Gap was lined on one bank or the other with Corps of Engineers main line levees.

The 1907 River and Harbor Act authorized a 6-foot channel on the upper river. The depth increase over the 4½-foot project was to be obtained by construction of rock and brush dikes, which, like the earlier dikes, were low structures extending laterally from the bankline into the river to constrict low-stage flows. Normally, the bankline opposite the dikes was protected with rock revetment or riprap to prevent erosion by currents redirected by the dikes. In addition, Lock and Dam 19 at Keokuk, Iowa was constructed as part of a hydro-electric facility in 1914, and Lock and Dam 1 at Minneapolis was completed in 1917 as part of the 6-foot channel navigation project (Degenhardt, 1973; Simons et al., 1974; Corps of Engineers, St. Paul, 1974).

As an indicator of the extent of dike construction under the 4½- and 6-foot projects, the locations of the 83 dikes that have been constructed in Pool 24 are shown in Figure 43. In this 27.8 mile reach of river more than 15 lineal miles of dike were built between 1879 and 1933. Forty-two percent of this effort was completed prior to 1900.

4.1.2 <u>The 9-Foot Channel Project</u>

In 1927, in response to increased traffic and a demand for deeper draft vessels on the river, Congress authorized the Corps of Engineers to obtain and maintain a 9-foot deep, 300-foot wide channel within the Mississippi River from St. Louis to Cairo. Adequate channel depths for the earlier 8-foot channel on the Middle Mississippi had proved difficult to obtain and maintain, particularly in the crossing sections between bendway pools. As a result dredging had been required on many of the crossings. It was assumed that a 9-foot minimum depth channel could be obtained through the construction of additional contraction dikes to constrict the river to widths ranging from 2500 to 2000 feet. By 1944 most of this contraction work had been completed; however, dredging was still required to maintain project depth.

In 1930, after extensive studies by the Corps of Engineers of the Upper Mississippi's potential as a modern transportation artery, Congress authorized the extension of the 9-foot channel project to include the river from the mouth of the Missouri to St. Paul. However, the approach authorized by the River and Harbor Act of 1930 was radically different from the contraction efforts of the 4½- and 6-foot channel projects. The authorizing legislation provided for a 9-foot deep, 300-foot wide navigation channel to be achieved by construction of a system of locks and dams to regulate the flow, as well as supplemental dredging.

This departure from the earlier contraction methods of navigation improvement stirred considerable controversy along the river, and to a degree this controversy continues today. Proponents of the project cited national defense and national economic growth and progress in their arguments. Opponents generally cited environmental concern. For example, the "Voice of the Outdoors," a column in the Winona Republican Herald, argued on 26 July 1930:

"We are still against the alleged 9-foot channel under the dam form of construction. We are now more firmly convinced than ever that it will be a gigantic commercial failure and will be impossible to maintain without spending millions of dollars each year in dredging operations. It will completely destroy bass fishing on the river and will look like a lot of link sausages on a map and smell worse than said sausage if they were left exposed to the present heat for a week. The scenic attraction of the river will be completely wiped out." (Corps of Engineers, St. Paul, (1974.)

On the positive side of the environmental ledger, the United States Bureau of Biological Survey conducted a study of the biological effects of Lock and Dam 19 at Keokuk and concluded:

"It is very probable that considerable portions of the Upper Mississippi River Wildlife and Fish Refuge would be benefited by the construction .... Immediately following the construction of any system of dams flooding the lowlands, an adverse period must be anticipated, but following the readjustment and re-establishment of the aquatic and marsh vegetation, the refuge should be an improved place for waterfowl and probably also for muskrats." (Corps of Engineers, St. Paul, 1974.)

The long term environmental impact of the system of locks and dams was evaluated in 1960 by Green in a report on ecological changes in the Upper Mississippi River Wildlife and Fish Refuge:

"The impoundment abruptly changed the river bottoms from an area of wide fluctuations in pool levels ranging from floods in the spring to drying out in the summer, to an area of semi-stabilized water in which, while spring floods still occur, the bottoms do not dry out in the summer. Thus, instead of wooded islands and dry marshes, we now have excellent marsh and aquatic habitat, with fairly stable water levels throughout the year... instead of drying up in the summer and winter, there is now water available throughout the year in the marshes, lakes, and ponds. Lack of marsh and aquatic plants is no longer a problem, and fish rescue is a thing of the past. Hay meadows and timbered areas are now in marsh, which offers excellent habitat for furbearers and waterfowl." (Corps of Engineers, St. Paul, 1974.)

Most of the locks and dams for the 9-foot project were constructed between 1930 and 1940. When viewed in longitudinal profile, the locks and dams form a series of "steps" in a "river stairway" as shown in Figure 44. River traffic ascends this stairway when moving upstream toward Minneapolis-St. Paul and descends when moving downstream toward St. Louis. The locks and dams regulate river flows to maintain the minimum 9-foot depth required for navigation. In Figure 44, the lower irregular line depicts the riverbed and the intermediate, stepped line indicates minimum pool water surface levels maintained by the locks and dams. The upper line depicts the water surface as it would appear under conditions of higher flow, that is when the gates of the dams are raised out of the water and the river is flowing in an "open" configuration.

The 9-foot project dams are low structures. This precludes economical water-power development or any flood control benefits. Flow through a dam is controlled by a combination of roller gates and tainter gates. Both types of gates, being movable, can be adjusted to control the amount of water passing through the dam, thus maintaining almost constant pool levels in times of normal and low flows. During the flood periods the gates are lifted entirely above the water level. The majority of lock chambers from the 9-foot channel navigation project are 110 feet wide and 600 feet long. In addition to the lock and gated sections, most structures also have earth dikes of varying lengths.

Most pools have a Primary Control Point (PCP) located about halfway upstream to the next lock and dam. At this control point water levels are maintained above a minimum elevation required for navigation and below a maximum elevation to minimize flooding as a result of navigation dam operation. By using this method of operation only the area between the control point and the downstream dam is inundated by the operation of the dam. The portion of the pool between the control point and the upstream lock and dam responds to variations in discharge essentially as the natural, open river would except that low-flow stages are held above the minimum elevation required for navigation.

Lock and Dam 26 at Alton, Illinois is representative of most of the locks and dams. It was built between 1934 and 1938, and consists of twin locks located adjacent to the Illinois bank and a gated dam extending from the locks to the Missouri bank. The twin locks were opened to traffic in 1938, and included a main lock 110 feet by 600 feet and an auxiliary lock 110 feet by 360 feet. The gated spillway, approximately 1,725 feet in length, consists of 30 tainter gates, 40 feet wide and 30 feet high, and three roller gates, each of which are 80 feet wide and 25 feet high (Armstrong, 1970).

Except for the upper and lower locks at St. Anthony Falls, this major project was completed in 1940, and with supplemental dredging it has provided a 9-foot channel from St. Louis to St. Paul. The permanent change in water levels submerged many of the dikes constructed during the 4½- and 6-foot channel projects, and as Green's report indicates, has dramatically changed the environmental character of the river.

On the Middle Mississippi efforts to attain the 9-foot channel project dimensions and eliminate maintenance dredging requirements continue today. Experience gained from the construction of regulating works by the Corps from 1927 to the World War II period indicated the need for additional contractive effort. Accordingly, an 1800-foot contraction plan was adopted for the Middle Mississippi. By 1960, it was evident that the 1800-foot contraction plan, which consisted of more than 800 timber pile dikes, was not capable of maintaining a 9-foot channel during low-flow periods. For a time, it was thought that this was due to the fact that many of the pile dikes had deteriorated and were losing their effectiveness. However, by 1965, numerous pile dikes had been converted to stone-fill dikes and still a dependable 9-foot channel had not developed.

The Corps was authorized in 1965 to construct a prototype reach (Figure 6) in a typical troublesome portion of the river (RM 140-RM 154), using a 1200-foot contraction width. The purpose was to develop additional empirical design criteria which would assure successful completion of the 9-foot channel project. Prototype reach construction was initiated in July 1967 and completed in March 1969. No dredging has been necessary within the prototype reach since its completion, and a preliminary evaluation indicates that the 1200-foot contraction is capable of developing navigation depths in excess of 9 feet.

To obtain a 9-foot channel with the least amount of contractive effort, reaches of the river are currently being contracted to a 1500-foot width. Preliminary investigation has revealed that the 1500-foot contraction width, with additional contractive effort at troublesome channel crossings, may be capable of achieving a dependable, year-round, 9-foot navigation channel (Degenhardt, 1973).

Dike construction programs in support of navigation projects on the Middle Mississippi have been extensive. Over 800 dikes with a total length of 91 miles have been projected out from the river banks into the channel. The location and number of dikes in a 16 mile reach, which includes the prototype reach, are shown in Figure 45. In addition, 122 miles of bankline revetment to prevent bankline erosion have been placed in the Middle Mississippi.

4.2 Geomorphic Response of the Upper Mississippi

4.2.1 River Position

The position of the Upper Mississippi in the Pool 24, 25, and 26 study reach was sketched briefly in Chapter 3 in regard to tributary influence on alluvial island formation. From Hannibal to Clarksville (Figure 5) the Upper Mississippi follows the western bluff line except for a few miles north of Louisiana, Missouri. Here the Salt River has built a deposit which forces the river into mid-valley for a short distance. As the neck of land separating the Mississippi and Illinois Rivers narrows toward the south, the influence of tributaries draining from the western bluff, such as the Cuivre River and Bob's, Peruque, and Dardenne Creeks, becomes increasingly significant. Below Lock and Dam 24 the Mississippi shifts across the valley to the eastern bluff line which it follows to its junction with the Missouri River.

A comparison of river bankline position on township plats surveyed between 1815 and 1830 with bankline position on the 1891 hydrographic charts indicates little change in bankline position. Thus, the 1891 charts are considered representative of the natural river position. Further comparison of the 1891 charts with more recent surveys and aerial photographs demonstrates that the position of the Upper Mississippi in this reach has not changed appreciably in 150 years.

Within the relatively stable banklines, however, there have been changes in the number, location, and configuration of alluvial islands. Figures 46 through 49 illustrate both the general bankline stability and island changes in several subreaches of the study area. In all cases, except where large islands have coalesced with the bankline (as in Figure 48 just upstream from Mosier Island), or where minor bankline erosion has occurred, the channel has maintained its historic position. A more detailed discussion of within channel changes is presented in the section on surface area. The conclusion here is that channel developments have not significantly affected the position of the Upper Mississippi River.

4.2.2 River Width

For consistency in this study, the river width is defined as the distance between vegetated banks taken normal to the general direction of flow in the river. Thus, the width is essentially the bankfull width of the river, and includes the width of the main channel and any side channels or islands in the section.

In some areas where measurements were made, the previously well-defined channel was obscured by higher water levels following closure of the locks and dams. For example the 1973 river widths were measured from aerial photographs taken at near bankfull stage, and since aquatic vegetation could not be distinguished from inundated terrestrial vegetation, the width was taken as the water surface width appearing on the photographs.

Of the three locks and dams in the detailed study area, the topographic configuration at Lock and Dam 24 is representative of conditions at many locks and dams on the Upper Mississippi. The proximity of the bluff line to the west of the river and the Sny Island Levee and Drainage District to the east of the river at Lock and Dam 24 strongly influences river width immediately upstream from the lock and dam.

River widths were measured at one-mile increments in Pool 24 and average values tabulated (Table 5). Examination of the maps and aerial photographs revealed that the most noticeable effects of the locks and dams occurred near these structures. Hence average values of width are presented over different reaches in the pools; that is the upper and lower fourths and the middle half. This stratification of the width data provides a reasonable basis for comparison of change in width between 1891 and 1927, the period of dike construction, between 1927 and 1940, the period of lock and dam construction, and the period 1940-1973 when the river adjusted to the presence of locks and dams.

Table 5 shows an average reduction of river width of 300 feet throughout Pool 24 during the period of dike construction. Between 1927 and 1940 all subreaches of Pool 24 show an increase in width, attributable, in part, to higher water levels in the pool behind the lock and dam which inundated the lower elevations of the floodplain. As the river adjusted to the locks and dams from 1940 to 1973, the immediate general increase in width was followed by a long term decrease in width in the upper 1/4 and middle 1/2 of the pool and an increase in width in the lower 1/4.

It is significant that where the lower 1/4 of a pool is not constricted by bluff line or levee along both banks of the river, such as in Pool 25 (Figure 49), major width changes occurred after construction of the lock and dam. In the case of Lock and Dam 25, average width in the reach immediately upstream from the dam increased from 4660 feet to 6950 feet during the period 1927-1973, as a result of inundation of the low-lying, unprotected floodplain to the east of the river. In contrast to an average 100-foot decrease in width in Pool 24 between 1927 and 1973, Pool 25 as a whole showed an increase from an average width of 5013 feet in 1927 to an average of 5776 feet in 1973.

In channels that are as stable as the Upper Mississippi, major changes in width are generally the result of abandonment of side channels and attachment of islands to the bank, or to deposition in dike fields. An excellent example is the middle half of Pool 25 (Figure 48) where the joining of two large islands with the east bank of the river just north of Mosier Island contributed to a decrease in average width of 270 feet between 1927 and 1973. Similarly, in the middle reach of Pool 24, Angle Island (RM 285) joined the Missouri (west) bank between 1940 and 1973. In 1927 Drift Island (RM 291) was a part of the Illinois floodplain. By 1940 a backwater channel had opened, creating an island again, but between 1940 and 1973 Drift Island rejoined the Illinois mainland. The effect of these major island changes on the middle reach of Pool 26 can be seen in Table 5 which shows a 300 foot increase in average width between 1927 and 1940 and a 500 foot decrease in average width between 1940 and 1973.

In summary, dike construction caused contraction of the Upper Mississippi between 1880 and 1927. The general effect of the locks and dams was to decrease the width below a lock and dam in both the upper 1/4 and middle 1/2 of the next pool. In the lower 1/4 of a pool a combination of levee and bluff line generally limits changes in channel width, but in an unrestricted location such as Lock and Dam 25, a significant increase in width followed construction of the lock and dam.

4.2.3 Surface Areas

To permit consistent measurements, the surface area of a river is taken as that area between the vegetated riverbanks. The surface area includes the area of the islands. Islands are defined as the vegetated areas within the channel banks and are separated from the mainland by the main channel and side channel. The riverbed area is defined as the surface area less the area of the islands, and consists of the area of the main channel and the area of the side channel.

The surface areas of the Mississippi River in Pool 24 are shown in Figure 50. Because the lengths of the subreaches in Pool 24 did not change in the last 100 years, the change in surface area has mirrored the change in river width. Thus, the surface area in Pool 24 decreased slightly during the period of dike construction, increased between 1927 and 1940, and then decreased again when Angle and Drift Islands coalesced with the floodplain.

In the lower 1/4 and the upper 1/4 of Pool 24 a discernible pattern of change emerges (Figure 50). In the upper reach, riverbed area has decreased while island area has increased through the period of record. Conversely, in the lower reach an increase in riverbed area has accompanied a decrease in island area.

The reduction in island area immediately upstream of Lock and Dam 24 reflects in part the influence of higher water levels which inundated the lower elevations of the islands. In the upper reach of Pool 24, just below Lock and Dam 22, the decrease in width noted in the previous section coupled with an increase in island area point to a lowering of bed elevation produced by a combination of contraction and clear water scour below the lock and dam. Lower bed elevations imply lower water levels and thus, an emergence of the lower portions of the islands.

The change of plan view area of the Mississippi River is illustrated in Figures 46 through 49. The major changes were submergence of islands and floodplain immediately upstream of the locks and dams and enlargement of islands below locks and dams. In these figures, new islands can be observed forming, enlarging, and merging with the floodplain in dike fields constructed in the peripheral areas of the channel.

The Crider-Cash-Pharrs Islands immediately above Lock and Dam 24 (Figure 46) provide an excellent example of decrease in island area in the lower 1/4 of a pool, and incidentally, illustrate the crab-claw shape so indicative of the processes of island building.

In the Clarksville-Carrol Island area immediately below Lock and Dam 24 (Figure 47) there has been a significant enlargement of all islands (1927-1973). Formation of new islands as well as chute channel abandonment can also be observed in this reach.

The Mosier Island reach in the middle half of Pool 25 (Figure 48) shows the growth of several major islands (1880-1927) just upstream of Mosier Island. These islands merged with the east bank (1927-1973) when several large chute channels filled. The resultant effect on river width has been discussed.

The Turner Island area just above Lock and Dam 25 (Figure 49) has experienced a major loss in island area since dam construction. Inundation of low-lying regions in this reach has also caused one of the most pronounced responses to man's activity to be found on the Upper Mississippi, a 150 percent increase in the average river surface width in the eight-mile reach above the lock and dam.

The evidence presented indicates some change in surface areas within the Mississippi channel during the period of dike construction, but the more significant changes have occurred since the period of lock and dam construction. The closure of side channels in Pool 24, 25, and 26 reaches is apparently a slow process requiring in some instances 30-40 years; however, from the data available it is evident that dikes have accelerated the processes of chute channel closure and island merger.

4.2.4 The Longitudinal Profile

The longitudinal profile of the Upper Mississippi from St. Anthony Falls to St. Louis is shown in Figure 44 together with the low-water surface before and after construction of the locks and dams. In the detailed study area of Pools 24, 25, and 26 elevations of the riverbed have been compared for 1891, 1930, 1940 and 1971. Changes in average riverbed elevation in the deepest 1000-foot width of river channel in Pools 24 and 25 are given in Table 6. Also changes in bed elevation in the thalweg are included in Table 6 for comparison. On the average, bed elevations in the thalweg are 6 feet lower than the average riverbed elevations in the deepest 1000 feet of river cross section. However, thalweg bed elevations in general vary in the same manner as average riverbed elevations, and so provide a good, readily obtainable indicator of trends in bed elevation.

During the period 1891-1930 the riverbed generally aggraded slightly; however, if the comparison is made between 1891 and 1940, a tendency toward long term degradation is apparent, except in the lower 1/4 of Pool 24. The response to locks and dams can be seen in the comparison of bed elevation change between 1940 and 1971. Pool 24 experienced a consistent pattern of aggradation, while Pool 25 continued to degrade except for slight aggradation in the middle reach. Thalweg bed elevations for 1891, 1930 and 1971 are plotted in Figure 51.

The lower 1/4 of Pool 24 and a portion of the upper 1/4 of Pool 25 are included. The aggradational tendency in Pool 24 between 1891 and 1930 is apparent. Between 1930 and 1970, aggradation immediately above the lock and dam and a consistent pattern of degradation and a major scour hole can be seen below the lock and dam. This is essentially the classic response to a dam that would be predicted by qualitative analysis using the Lane relation (Equation 11).

Considered without reference to local conditions, the increase in bed elevation during a period when dike construction resulted in decreased widths appears inconsistent, but not when the effects of development are viewed as a modification of existing natural tendencies in the Upper Mississippi. Both Lane's and Rubey's descriptions of the Upper Mississippi as more braided in character than meandering have been cited (Section 3.2.1.2). Further, Lane (1957) has attributed this characteristic to overloading as a result of conditions inherited from the Pleistocene, producing a slowly aggrading stream. Rubey (1952) also indicates that while the Upper Mississippi may scour its channel to bedrock at some locations during floods, it "may be very slowly aggrading." Viewed in the context of a long established natural process, the continuation of a slight tendency toward aggradation during the 40-year period of dike construction is not surprising.

In addition, the contraction dikes constructed on the Upper Mississippi were generally low structures in comparison to bankful stage and apparently exercised a major influence on the flow at only the lower stages. For example, prior to 1891, 41 wing dams were constructed in the study reach of the Upper Mississippi River, 15 of them in what is now Pool 25. These wing dams were all low dikes with crests at a level 6 feet above the 1864 low water and were constructed with rock, brush and sand. Using Pool 24 as an example, 60 percent of the dike construction took place after 1900 and 26,000 lineal feet of dike, 32 percent of the total effort, was built, repaired or extended after 1920. This effort, late in the period of dike construction, coupled with lock and dam construction evidently contributed to a reversal of a long term natural tendency toward aggradation.

The response to lock and dam construction has been a function of the pool considered. In Pools 25 and 26 the tendency toward degradation initiated in the 1930's has continued. In Pool 24, however, the river-bed in the deep part of the channel has aggraded approximately two feet since construction of Lock and Dam 24.

4.2.5 <u>The River in Cross Section</u>

Cross sections were plotted and measured at five-mile intervals in Pool 24 and at several locations in Pools 25 and 26. Sections were compared at the same location for the years 1891, 1930, and 1971. The effect of the period of dike construction on surface width, bed elevation, average depth and thalweg position are clearly evident. The change in bed elevation and average depth subsequent to lock and dam construction are also apparent. Sample cross sections for a geomorphically active reach of each pool are presented in Figures 52, 53, and 54, respectively. In each case sections are shown looking downstream with the east bank of the river on the left. Quantitative information developed from these sections is summarized in Table 7.

In Pool 24, the Cottel Island section (Figure 52) is located at river mile 300 just downstream from Lock and Dam 22. Between 1894 and 1896 several dikes were placed along the left bank of the river just upstream from this section, and between 1918 and 1930 additional dikes were built along the western bank of Cottel Island (Figure 43-RM 300). About 20,000 cubic yards of material was dredged from the channel just upstream from this reach between 1936 and 1937. Subsequent dredging has all been downstream in the Taylor crossing and Gilbert Island reach (RM 298). The sequence of aggradation between 1891 and 1930 followed by degradation is reflected in the average depth in Table 7 and can be observed in the main channel in Figure 52. The marginal effectiveness of the early dikes in this reach as well as significant contraction following the second phase of dike construction and installation of Lock and Dam 22 also are apparent. Although Pool 24 has generally experienced aggradation since lock and dam construction, this section shows obvious degradation in the main channel. Contributing factors to this trend are the section's location just downstream from Lock and Dam 22, extensive contraction effort in the reach, and frequent dredging. The combined effect of contraction and dredging on section shape is examined in detail in Chapter 5.

The Eagle Island reach is located at river mile 270 in Pool 25, 3.5 miles downstream of Lock and Dam 24 (Figure 53). Dike construction along the west bank of Clarksville Island (Figure 47) just upstream from this section was accomplished primarily between 1924 and 1930; however, several long dikes had been placed opposite Eagle Island in 1894 and 1890. More than 365,000 cubic yards of material was dredged from this reach between 1925 and 1935, and another 126,000 cubic yards was removed in 1949. Repeated dredging of major quantities of material in the vicinity of Amaranth Island (Figure 47) just downstream from Eagle Island indicates that this general reach has posed chronic problems for navigation. The average depth in this section decreased slightly (Table 7) between 1891 and 1930 as the single channel of 1891 was divided into two channels by a wide submerged bar. Again, the early dikes apparently were only marginally effective. By 1971 average depth in the section had increased and the main channel was deeper and more concentrated than in 1891. Although, the second phase of dike construction accomplished considerable contraction, the chute channel east of Clarksville and Carrol Islands remained open. Since large quantities of dredged material have been available for disposal in the dike fields in this reach, the possible influence of dredging operations in restructuring the cross section cannot be overlooked. Again this influence is examined in detail in Chapter 5.

In Pool 26, the Turkey Island reach (Figure 54) is located adjacent to Cuivre Island (RM 237) and the Cuivre River. This reach has experienced significant morphologic change in the last 75 years. In 1899 a dike field consisting of eight dikes, each in excess of 1000 feet long, was constructed along the east bank of the river opposite Turkey and Cuivre Islands. This field was supplemented by four additional dikes placed along the east bank of Turkey Island and in the Turkey Island chute between 1910 and 1919. Dredging adjacent to Turkey Island was not required until 1949, but since then, more than 660,000 cubic yards of material have been dredged from this reach. In 1891 the section consisted of two deep channels, one along the east bank of the river and one adjacent to Turkey Island. By 1930 the average depth had decreased slightly (Table 7) and the channel had developed an almost braided character with three deep channels separated by shallow bars. The dike field opposite Turkey Island effectively closed the previously existing deep channel along the east bank of the river. Subsequent to 1930 the average depth in the section almost doubled and the multiple channels of the past were replaced by a single main channel.

Analysis of these cross sections supports the conclusions derived from previous examination of width, area, and bed elevation changes. The apparent limited effectiveness of dikes constructed during the 1890's is significant. This limited effectiveness of early dike construction, coupled with the concentration of effective dike construction effort in the latter part of the 1891-1930 period, certainly contributed to the observed continuation of a long term natural tendency toward aggradation through the 40-year period of dike construction. Immediate response would not be expected and observations indicate that the lowering of bed elevations did not occur until during and after the era of lock and dam construction.

4.3 Hydraulic Response of the Upper Mississippi

In the detailed study area of Pools 24, 25, and 26, geomorphic analysis has shown that although the position of the Mississippi through this reach has been relatively constant since 1891, there have been major changes in island area and in riverbed area. In addition, the response of the river to development has resulted in significant alteration of bed elevations and flow area. These geomorphic changes are reflected in trends in the stage and discharge record.

4.3.1 Gaging Stations

The Pool 24, 25, and 26 reach is bracketed by three gaging stations with relatively long term discharge and stage records. The Alton, Illinois station (Figure 5) just below Lock and Dam 26 has reported discharges and stages intermittently from 1844 to 1896 and then continuously to the present. At Keokuk, Iowa (Figure 1), 65 river miles above the detailed study area, the discharge record is discontinuous from 1851 to 1880 and continuous thereafter, while maximum and minimum stages have been reported intermittently from 1851 to 1870 then continuously to the present. In addition, the Corps of Engineers has compiled stage records at Hannibal, Missouri, 9 river miles above Lock and Dam 22, and at Grafton, Illinois at the confluence of the Mississippi and Illinois Rivers (Figure 5). Discharge and stage data from these 4 stations provide an excellent long term indication of hydraulic changes in Pools 24, 25, and 26. It should be noted that both the Alton gage and the Keokuk gage are located between a lock and dam and a major tributary, the Missouri River at Alton and the Des Moines River at Keokuk. Consequently, both gages are influenced to a degree by backwater.

4.3.2 Discharge and Stage Trends

The annual maximum, mean and minimum discharges for Alton and Keokuk are given in Figures 55 and 56. The annual maximum and minimum stages at Alton, Keokuk, Hannibal and Grafton are shown in Figures 57 through 60 respectively. The results of a statistical analysis which was applied to the time series of discharge and stage data to determine significant trends, are summarized in Table 8. Where a trend was not discernible, the character of the data was termed "none" and a mean value line was drawn through the data. Where an upward or downward trend in discharge or stage with respect to time was determinable, a trend line through the data is provided on the appropriate figure.

The annual flood discharges at Alton and Keokuk have remained on the average unchanged in the last 110 years. The mean annual flow has increased slightly at Alton but decreased slightly at Keokuk. The annual minimum discharge has increased at both gages.

On the average, the annual maximum stages at Alton, Keokuk, and Grafton have remained unchanged through the 100 years of record. In Pool 22 at Hannibal, however, the annual maximum stage shows a definite uptrend. On the assumption that the same processes are at work in Pool 22 as have been observed in Pool 24, this uptrend could represent response to the combined effects of decreasing width and increasing bed elevation through 1930. Subsequent to 1940 continued increase in maximum stage during a period of lowered bed elevations can be attributed in part to the compensating effects of decreased river widths in most of the pool following lock and dam construction.

The minimum stage records strongly reflect man's development of the Upper Mississippi. First full pool was reached in Pool 26 in 1938 and the immediate increase in minimum stage is apparent in Figure 60. This same response can be seen at Hannibal in the late 1930's following construction of Lock and Dam 22 (Figure 59), and at Alton (Figure 57) in the early 1960's when Dam 27 below St. Louis was completed. The minimum stage records at both Alton and Keokuk show a definite decrease following lock and dam construction in 1914 and 1938, respectively, which can be attributed to degradation immediately below each lock and dam.

4.3.3 Sediment Discharge

The Upper Mississippi River has been described as a clear water stream in comparison to the Middle Mississippi. On the upper river at

Hannibal, just above the detailed study area, suspended-sediment samples have been collected since 1943. Average suspended-sediment discharge for water years 1949-1963 has been 56,000 tons per day or 20,400,000 tons per year (Jordan, 1968). This represents a little over 10 percent of the suspended sediment load of the Middle Mississippi at St. Louis. For comparison, the average daily sediment load at St. Paul is about 500 tons per day (Mack, 1970). Suspended sediment discharge data at Hannibal between 1949 and 1963 is summarized in Table 12 and compared with this same data for the Missouri River and the Middle Mississippi.

In the Mississippi at Hannibal the suspended sediment includes very little sand. The average of all particle-size analyses between 1951 and 1962 showed only 2 percent sand, with few samples deviating significantly from 2 percent (Jordan, 1968). Measurements and estimates by experienced observers indicate that for streams in the Upper Mississippi Basin the bed load generally represents about 10 percent of the total sediment load (Mack, 1970). This figure appears reasonable in the light of Lane and Borland's guidelines (Table 3).

The significant increase in sediment yield from the northern portions of the Upper Mississippi Basin to the southern portions of the basin (Figure 42) was used as a basis for selecting the Pool 24 - Pool 26 reach as a detailed study area. More specifically, from Wabasha, Minnesota to Hannibal, Missouri, the estimated long-term annual rate of sediment production increases from 4 to 181 tons per square mile. This increase is due primarily to changes in land use, annual runoff, soil type and topography. For the drainage area as a whole, the proportion of land in cultivation increases by 30 to 40 percent between these two points, and the runoff per square mile is also markedly increased (Mack, 1970).

Sediment records are generally not of sufficient length to permit an evaluation of the effects of man's activity on sediment transport in the Upper Mississippi. In addition, the large natural variability in annual sediment discharge makes the detection of meaningful trends difficult. However, in conjunction with the Upper Mississippi River Comprehensive Basin Study (1972) the few long-term records that are available were reviewed and tested for trends in yield. This analysis, showed that, while a trend was not discernible in all areas, in some areas a decrease in sediment yield was evident.

The construction of the navigation locks and dams on the Upper Mississippi has altered the movement of sediment through the river system. For example Pools 2 and 3 and Lake Pepin (Pool 4) serve as effective traps for most of the sediment reaching the Mississippi above Lake Pepin (Figures 1 and 44). It is estimated that only about 11 percent of the total amount of sediment entering these pools reaches the outlet of Lake Pepin (Corps of Engineers, St. Paul, 1974). This trapping of sediments by each of the navigation pools of the system is certainly a contributing factor to sediment deficiency and resulting degradation in the lower pools of the navigation system.

4.4 Geomorphic Response of the Middle Mississippi

4.4.1 River Position

The Middle Mississippi River has been well behaved in comparison to its extension to the south, the Lower Mississippi, where large scale morphologic change on the natural river has been the norm (Figure 7). Humphreys and Abbot observed in 1861 that the Middle Mississippi maintains a position along the western bluff line for most of its length between St. Louis and Thebes Gap, as is true today (Figure 6). In general, the floodplain to the east has not been affected by serious encroachments of the main channel. In this reach the river has shown no obvious tendency to meander (Simons et al., 1974).

The Middle Mississippi has, however, experienced several major local changes in its alignment since the river was first mapped. One of the most notable occurred in 1881 when the Mississippi River broke into the Kaskaskia River nine miles above its confluence with the Mississippi (Figure 61). Since the Kaskaskia route to the confluence was shorter than the Mississippi route, the main flow of the Mississippi followed the Kaskaskia channel, and the former Mississippi channel gradually filled with sediment. St. Marys, Missouri, which had been on the Mississippi was left five miles distance from the river (Tiefenbrun, 1963).

A comparison of bankline maps of the Middle Mississippi between 1818 and 1968 by Simons, Schumm, and Stevens (1974) shows that the channel position has been quite stable for the last 150 years, with the exception of local changes such as the Kaskaskia cutoff. As with the Upper Mississippi the position and size of islands within the channel have changed with time. An excellent example of relative bankline stability with significant in-channel change is given by the time lapse comparison of the Devil's Island reach in Figures 26a-d.

4.4.2 River Width

In conjunction with a geomorphic study of the Middle Mississippi (Simons et al., 1974) river widths were measured at one-mile increments between Jefferson Barracks (RM 170) and Cairo (RM 0). This data, as summarized in Table 9a, shows that the average width was about 2000 feet greater in 1888 than in 1821. Dike construction on the Middle Mississippi was initiated in 1881 and has continued to the present. The effect on river width is apparent in the 2100 foot decrease in average width between 1888 and 1968. This decrease in river width and its relation to dike construction is evident in Figure 45 which shows dike locations superposed on the 1889 and 1970 banklines between RM 138 and RM 154.

Maher (1963) presents evidence that the river widened from natural causes in the vicinity of St. Louis between 1808 and 1849. This rapid widening, which is evident in Table 9b, resulted in serious deterioration of the St. Louis harbor. In 1838, based on a plan by R. E. Lee, the federal government initiated work to correct the problem by constructing a series of dikes from the Illinois shore to confine the river to a definite channel. These dikes reduced the river width by half, and since that time the bankfull width at St. Louis has been 2100 feet (Table 9b).

It is possible that the large floods which occured between 1844 and 1888, or a combination of large floods and floodplain development could have caused the widening of the Middle Mississippi in the St. Louis reach. In that period there were four floods that equalled or exceeded 1,000,000 cfs (Simons et al., 1974).

4.4.3 Surface Areas

As with the Upper Mississippi, the length of the Middle Mississippi has not changed appreciably in the last hundred years. Thus, surface area change has mirrored change in river width. Using the same definitions of surface area, island area, and riverbed area as for the

Upper Mississippi, surface area changes on the Middle Mississippi between Jefferson Barracks and Cairo are summarized in Table 10. Since 1888, during a period of dike construction and bankline revetment, the river surface area has been reduced by almost 40 percent, the island area by one-half, and the riverbed area by more than one-third. Both the decrease in surface area and loss of island area are apparent in Figures 25 and 45. The loss of a large alluvial island such as Powers Island (RM 36) in Figure 25 or Calico Island (RM 148) in Figure 45 has a major impact on the riverbed area to surface area ratio in a reach.

4.4.4 The Longitudinal Profile

The pattern of change in riverbed elevations has not been as complex on the Middle Mississippi as that observed on the Upper Mississippi. Riverbed degradation has occurred along the Middle Mississippi wherever the river channel has been narrowed. In the general case, degradation is the expected natural consequence of reducing width, increasing the flow per unit of width, and thus, increasing the transport capability of the water per unit of width (Simons et al., 1974).

Supported by a detailed analysis of dike construction and its effectiveness in the RM 138 - RM 154 reach (Figure 45), Degenhardt (1973) concluded that the permeable dikes constructed in the high sediment transport environment of the Middle Mississippi below the Missouri between 1889 and 1907 were quite effective. In fact,

> "With the exception of further narrowing of the river in some localities and the reduction of flows through certain side channels, the changes in width were generally not so great during the 63-year period from 1907 to 1970 as compared to the previous 18-year period."

The contraction achieved with the early phases of dike construction on the Middle Mississippi apparently overrode any existing natural tendency toward aggradation and produced a consistent pattern of degradation through the era of dike construction.

The riverbed elevations in the 14-mile reach from RM 140 to RM 154 are shown in Figure 62. Here the average bed elevation is the mean elevation in the low water channel, determined as the average of from 15-20 riverbed elevations at a cross section. The 1889 riverbed elevations describe the level of the riverbed in its natural state. The river in this 14-mile reach was about 4800 feet wide in 1889. By 1966 the river had been contracted to an average width of 1800 feet. The riverbed had lowered about 8 feet between 1889 and 1966. As previously indicated, the Corps selected this reach as a prototype study reach in July 1967. Between 1967 and 1969, this test reach was narrowed from 1800 feet to 1200 feet in width. The 1971 bed profile is shown in Figure 62. The contraction from 1800 feet to 1200 feet resulted in a 3-foot lowering of the riverbed. In 1971 the low-water riverbed in the 14-mile reach between Mile 140 and Mile 154 was on the average 11 feet lower than in 1889 (Simons et al., 1974).

4.4.5 The River in Cross Section

The combined effect of decreased width and lowered bed elevations on the Middle Mississippi is apparent when river cross sections are considered. For example, the change in cross-sectional geometry at St. Louis is shown in Figure 63 and can be compared with changes in river width of Table 9b. In 1837 the river at St. Louis was 3700 feet wide and had an average depth of 30 feet at bankfull stage. Dike construction between 1830 and 1888 decreased the width permanently to

2100 feet. In 1973 the average depth prior to the 1973 flood was about 45 feet at bankfull stage and the width-to-depth ratio had been decreased from 123 to 45.

The cross-sectional area at bankfull stage in 1973 was approximately 80,000 square feet as compared to 120,000 square feet in 1837. Narrowing of the channel at St. Louis has reduced the bankfull channel area by about one-third. A similar decrease has occurred throughout the Middle Mississippi wherever the river channel has been contracted (Simons et al., 1974).

Thus the response of the Upper Mississippi River to development for navigation using contraction dikes has been different than that of the Middle Mississippi River. There are two reasons for the different response to dikes in the Upper and Middle Mississippi Rivers. First, the dikes in the Upper Mississippi were low in comparison to the bank-full stage. The high dikes of the Middle Mississippi are more effective in concentrating all flows in the navigation channel. Second, the amount of sediment transported in the Middle Mississippi is much greater than that transported in the Upper Mississippi. The combination of larger sediment loads and high dikes in the Middle Mississippi River transformed a part of the former river channel section into flood-plain thus decreasing the channel capacity at large flows.

4.5 Hydraulic Response of the Middle Mississippi

4.5.1 The St. Louis Gage

Water discharge and stage measurements at St. Louis have been made intermittently between 1843 and 1861, and continuously thereafter. The flood peak discharge of record at St. Louis was 1,300,000 cfs recorded in 1844, with almost 70 percent of this being contributed by the

Missouri River. For comparison, the largest recorded flood on the Upper Mississippi was 573,000 cfs at Alton in 1858. The minimum discharge at St. Louis was 18,000 cfs which occurred in 1863.

Unlike the Upper Mississippi gages at Keokuk and Alton, the St. Louis gage is not influenced by either the backwater effects of a major tributary or the construction of locks and dams. However, flows and stages on the Middle Mississippi are influenced by two factors not present on the upper river, the construction of Corps main-line levees along the river, and the construction of storage dams on the Missouri.

In general, levee construction results in a decrease in floodplain storage and produces increased flow discharges for discharges greater than bankfull stage. For example, the 1844 peak discharge passed St. Louis at a 41.3 foot stage. It has been estimated that, due to construction of contraction works and the levee system, this same discharge would now pass St. Louis at approximately a 52.0 foot stage (Simons et al., 1974). In regard to storage dams on the Missouri, the first structure of this type was not completed until 1940. Although the effect of these reservoirs on the flow at St. Louis depends on the method of reservoir system operation, in general decreased maximum flows and increased minimum flows can be anticipated.

4.5.2 Discharge and Stage Trends

The average annual maximum, mean and minimum discharges at St. Louis are given in Figure 64. On the average, the discharge of present day peak floods is slightly lower than in the past, while mean annual discharge has not changed in 130 years. The average annual minimum flow has increased slightly during the period of record. The discharge record at St. Louis supports the conclusion that the cumulative effects of man's development together with the effects of factors such as land use or climatic change have not significantly altered average annual flows on the Middle Mississippi.

The annual maximum and minimum stages on the Market Street gage at St. Louis are shown in Figure 65. The annual maximum stage has increased slightly during the 130 years of record, and variations in the annual maximum stage are greater now than in the past. The highest recorded stage at St. Louis was 43.3 feet in 1973. The trend of annual minimum stages has been downward during the period of record. On the average, minimum stages are 6 feet lower than in the period 1860-1870.

Geomorphic comparisons have shown that as the Middle Mississippi was deepened for navigation by decreasing the width with rock and pile dikes, a significant decrease in cross-sectional flow area has resulted. In most cases on the Middle Mississippi the construction of a dike field has resulted in the development of a new bankline displaced channelward from the original bankline. This evolutionary sequence is described in Chapter 3 and can be seen in Figure 28. In addition, levee construction has isolated much of the floodplain from the river channel and reduced overbank storage.

The effect of the decrease in both flow area and overbank storage can be seen in the increased maximum stages of Figure 65 which have accompanied a downward trend in maximum discharge. The hydraulic response of the low flow river to lower bed elevations can be seen in the decreased minimum stages that have accompanied an increase in minimum discharge.

4.5.3 Stage Versus Discharge

The effect of man's development on the discharge-stage relation can be seen most clearly in a comparison of peak flood discharges and their related stages. The 10 largest flood discharges and the highest 10 stages for the indicated period of record are ranked in Table 11. The discharge and stage ranks of the 1973 flood are significant. While ranking only 10th in discharge, the 1973 flood produced the St. Louis record stage of 43.3 feet. In addition, four discharges in recent years (between 1943 and 1951) which did not have flows large enough to rank in the top-ten discharges produced stages which ranked 3rd, 4th, 5th, and 6th for the period of record.

Simons, Schumm, and Stevens (1974) analyzed the change in stage at St. Louis for similar discharges on the natural 19th century river and the present-day developed river. The trend of changing stage for all discharges at St. Louis based on this analysis is shown in Figure 66. For example, the records show that the 1973 discharge of 855,000 cfs produced a stage of 43.3 feet. Ninety years before, in 1883, a similar discharge of 863,000 cfs produced a stage of 34.8 feet. The increase in stage from the natural to the developed river for this discharge is 9.5 feet, and is plotted as +9.5 feet at .85 million cfs in Figure 66. For all flows above 300,000 cfs stages are now higher, while for flows below 300,000 cfs, stages are lower now than on the natural river. The average annual peak flood at St. Louis is 500,000 cfs.

4.5.4 Sediment Discharge

Almost 90 percent of the suspended sediment load of the Middle Mississippi at St. Louis is contributed by the Missouri River.

Suspended sediment samples on the Missouri at Herman, Missouri, have been collected since 1948. Suspended sediment discharge for water years 1949 through 1963 have averaged 425,000 tons per day, or 155,000,000 tons per year. At St. Louis acquisition of sediment data on a scheduled basis also began in 1948. The average suspended sediment discharge for water years 1949-1963 has been 441,000 tons per day or 160,900,000 tons per year (Jordan, 1968).

Analysis of the suspended sediment at St. Louis shows a composition of about 85 percent silt-clay and 15 percent sand. Based on the guidelines of Lane and Borland (Table 3) a reasonable estimate of the contact load is 8 to 10 percent of the measured suspended sediment load. In regard to bed-material size at St. Louis, Tuttle (1970) notes that median diameters of bed-material samples are variable with time, being quite sensitive to the magnitude of the mean annual flow. For example, in high flow years median diameters range from .6 to 1.1 mm, while in low flow years, samples average about .3 mm in size. In general, most of the bed material in the main channel falls in the range .125 to 1.0 mm (Degenhardt, 1973).

Table 12 provides a summary of the suspended-sediment at Hannibal, Herman, and St. Louis. In regard to these quantities, Jordan (1968) noted that the combined flows of the lower Missouri and Upper Mississippi generally transport more suspended sediment, sand in particular, than does the flow of the Mississippi at St. Louis. This results in an excess of sediment between Herman, Hannibal, and St. Louis in most years. Although the sediment record at St. Louis is not of sufficient length to permit a "before development" and "after development" comparison, an examination of the record indicates a trend toward decreasing amounts of sediment moving in the Middle Mississippi since 1965 (Simons et al., 1974).

4.6 Summary and Contrasts

Development for navigation on both the Upper and Middle Mississippi between 1890 and 1930 followed the same theme of contraction, revetment, and dredging where necessary to attain desired depth. After 1930 the Corps continued to employ open river training works and dredging on the Middle Mississippi to achieve a 9-foot channel. On the upper river, however, the basic approach was changed to one of flow regulation by a system of navigation locks and dams, supplemented by dredging. While the response of the Middle Mississippi has been predictable, the response of the Upper Mississippi has been quite complex.

An analysis of river position based on a time-sequenced comparison of river banklines leads to the conclusion that the Mississippi between St. Paul and Cairo has not changed its position appreciably in the last 150-200 years. Using the change in river position as an indicator of stability, both the upper and middle rivers are quite stable. In terms of degree of stability, several significant local changes in position on the Middle Mississippi, such as the Kaskaskia cutoff, support the characterization of the Middle Mississippi as somewhat less stable than the Upper Mississippi, but certainly far more stable than the Lower Mississippi.

The use of dikes to create a navigation channel produced a slight decrease in width between 1890 and 1930 on the upper river and a major decrease in width between 1890 and the present on the middle river. Although detailed conclusions relative to geomorphic and hydraulic change in specific reaches on the Upper Mississippi subsequent to 1940 require an analysis of the particular pool in question, general trends in Pools 24, 25, and 26 appear reasonably representative of changes on the upper river following lock and dam construction. The immediate response to lock and dam construction was an increase in surface width throughout a pool; however, the long-term response has been a decrease in width immediately below a lock and dam and a slight increase in width just above a lock and dam.

The entire Mississippi above Cairo has experienced considerable within-channel change. These changes are reflected in variations in surface area, island area and riverbed area. Because the length of the Mississippi above Cairo has not changed appreciably, surface area change has generally mirrored the change in river width. Dike construction on the Middle Mississippi has produced significant decreases in island area and in riverbed area. On the Upper Mississippi, again, the response was more complex and a function of position in a pool. Higher water levels immediately upstream of a lock and dam have produced decreased island area, while a lowering of bed elevations downstream from a lock and dam has resulted in lower stages and increased island areas.

Bed elevations on the Middle Mississippi have been lowered throughout the period of dike construction. In a 14-mile reach examined in detail, the riverbed had lowered almost 11 feet between 1889 and 1966. The period of dike construction on the upper river (1890-1930) was one of slight aggradation of the riverbed. The limited effectiveness of the low dikes constructed on the upper river, and the concentration of construction effort toward the end of the era of dike construction, coupled with a natural tendency toward aggradation on the Upper

Mississippi contributed to this pattern of increasing bed elevations. This trend was reversed between 1930 and 1940 in the three pools of the detailed study area and general degradation has continued to the present in the lower two pools. Pool 24, however, has experienced general aggradation since 1940, indicating a tendency to trap incoming sediments from Pool 22 and the Salt River and create a sediment deficient condition in the downstream pools. Degradation has not been of the same magnitude as on the Middle Mississippi. Local exceptions to these trends include some aggradation immediately above locks and dams and local scour below.

Viewing the river in cross section provides an integrated picture of the effects of changes in width, surface area, and bed elevation. In particular, the response to dike construction is clearly evident in the cross-sectional view. Flow area at St. Louis has been progressively decreased until it is now only two-thirds that of the natural river. A similar decrease has occurred all along the Middle Mississippi wherever the channel has been contracted. On the Upper Mississippi flow areas generally decreased during the period of dike construction and increased following lock and dam construction in response to the variation in surface width and bed elevation.

Geomorphic response of the Mississippi above Cairo to man's activities is reflected in the hydraulic parameters of discharge and stage. Annual peak flood discharges on the upper river have remained, on the average, unchanged through the period of record. On the middle river present day peak floods are, on the average, slightly lower than in the past, reflecting the construction of storage dams on the Missouri. Minimum flows have increased slightly both above and below the mouth of the Missouri. The effect on river stage has been more significant. At St. Louis, the decrease in both flow area and overbank storage has contributed to an increase in the annual maximum flood stage. Although present day floods on the Middle Mississippi produce flood stages higher than similar discharges produced in the past, levees prevent flood damage when the river exceeds bankfull stage. Under natural conditions, flood damage occurred whenever the river exceeded bankfull stage.

On the Upper Mississippi minimum stages have been strongly influenced by man's development. Minimum stages have decreased at locations immediately below a lock and dam and have increased sharply at locations above a lock and dam shortly after first full pool was reached at each location.

Sediment data supports the characterization of the Upper Mississippi as a clear water stream and the Mississippi below the Missouri as a heavy sediment carrier. A little over 10 percent of the suspended sediment load at St. Louis is contributed by the Upper Mississippi. While 15 percent of the suspended load at St. Louis is sand, sediment data indicates that very little sand is moving in suspension in Pools 24, 25, and 26.

Sediment records on the Mississippi above Cairo are not of sufficient length to permit an accurate determination of the effect of development on sediment load. Available data does suggest that sediment loads have been decreasing in the recent past. The sediment trapping effect of the upstream pools of the Upper Mississippi lock and dam system have certainly been a contributing factor to the observed general degradation in the lower pools of the system.

This analysis of the geomorphic and hydraulic response of the Mississippi above Cairo to man's activity completes the development of the necessary baseline against which the effects of dredging operations on a river will be studied. The geologic history of the study area as well as a concept of the conditions that existed on the natural river have been essential to this analysis, and also support the analysis of dredging effects. A general understanding of basic principles of river mechanics is necessary to analyze any river engineering problem. In addition, a detailed understanding of those aspects of river mechanics and morphology that bear directly on the dredging and disposal problem is essential to this study. The examination in Chapter 3 of the morphology and hydraulics of the crossing and pool sequence, the island and side channel configuration, and basic concepts of bed-material transport reflects this requirement.

Chapter 5 addresses the specific objectives of this study. In Chapter 5 the interaction of dredging operations with river morphology and hydraulics is investigated with emphasis on the response of the Upper and Middle Mississippi River to the dredging process. In addition, the problem of open water disposal of dredged material in the river environment is viewed against the background of the morphology and hydraulics of the river.

Chapter 5

GEOMORPHIC AND HYDRAULIC RESPONSE OF THE RIVER TO DREDGING

5.1 The Role of Dredging

Dredging is defined as a process by which sediments are removed from the bottom of streams, lakes, and coastal waters, transported by ship, barge, or pipeline, and discharged in open water or on land. To the river engineer concerned with maintaining navigable depths in a waterway system, dredging is only one of many possible solutions to the problem. On the Mississippi River the "permanent" solutions to the navigation problem discussed in previous chapters include contraction by dikes and revetment, and flow regulation with locks and dams. These permanent solutions, however, have not succeeded in eliminating the requirement for the dredging of significant quantities of sediment to maintain the desired navigation channel.

If contraction and regulation are viewed as permanent solutions to the problem of maintaining navigable depths in a river, dredging must be viewed as a temporary solution to the problem. Case histories confirm the temporary nature of the dredging process, but experience also indicates that dredging has generally been a necessary adjunct to navigation improvement programs.

As an alternative to providing navigable depths, dredging can be compared to such solutions as bandalling and the placing of guide vanes or bottom panels to guide the current so as to increase erosive action in the channel. A bandal, as constructed in India for example, is a frame, consisting of locally available material such as bamboo stakes, planted in the riverbed. These stakes, when connected by horizontal ties and strengthened by sloping stakes, form a linear frame on which bamboo screens are hung. Bandal screens are placed in fields along both sides of the river, oriented at 30 to 40° toward the main current and tend to direct the current into the main channel (Figure 67a). Bottom panels have been employed successfully in the Soviet Union. Here stakes are driven into the riverbed and vertical vanes of wood, metal, or mat are attached extending from the bottom to mid-depth. With bottom panels the purpose is to induce a spiral flow pattern and increase erosion in the main channel, so panels are oriented from 15 to 20° into the flow (Figure 67b). As the construction of panels is relatively expensive, dredging may provide a more economical solution. In some instances a combination of bottom panels and dredging has been successful in maintaining navigable depths across a bar (NEDECO, 1959, 1973).

Temporary solutions to the navigation problem have the advantage of being relatively simple and direct in their application. They also afford a degree of flexibility in meeting unexpected or changing requirements in a waterway system not possessed by more permanent solutions which require large capital investment and long range planning. Temporary solutions to the navigation problem, to include dredging, suffer from the serious disadvantage that they treat the symptoms and not the disease. Bandals, panels, and dredging all attempt to change the local configuration of the channel without changing the forces that have produced that configuration, that is, the general patterns of water flow and sediment transport. Thus, changes resulting from these methods can be expected to prevail against the dominant forces of the system for only a limited period of time.

The early dredging records of the Memphis District on the Lower Mississippi illustrate the temporary nature of channel improvement by dredging. Although more than 73,000,000 cubic yards of material were moved by the dredges in this District since the initiation of dredging in 1895, the low-water seasons of 1930 and 1931 found the channel in much the same condition as in 1895. No permanent results could be noted. The river had been kept open for navigation by boats of greater draft and tonnage but no permanence of channel or assurance of adequate depths and widths had been obtained. The results were, as expected by the early engineers, temporary in their nature but effective in keeping the channel open (Somervell, 1932).

In this chapter a brief summary of the development and use of dredges on the Mississippi River is followed by an analysis of the stability of a dredged cut in alluvium, based on both field data and hydraulic model studies. The influence of dredging on river morphology and hydraulics as well as the use of the dredging process as an agent for morphologic change are examined. Finally, the impact of dredging on the 9-foot channel project and the problem of dredged material disposal are investigated.

5.2 Dredges and Dredging Operations

5.2.1 Dredge Development on the Mississippi

The devices which have been used on the Mississippi for dredging can be grouped into several categories based on the process involved. These include: stirring and scraping devices, current deflectors, mechanical devices, jets, and the suction dredge. A proposal to equip rivercraft with hydraulic jets to make them independent of any general improvement of the channel is cited in the Introduction; and, indeed, water jets were successfully used in the vicinity of St. Louis in 1881 to provide an increase of several feet in depth across a bar. In 1896 a jet dredge was constructed and used with some success on short bars between St. Louis and Cairo (Ockerson, 1898).

The first devices employed for dredging on the Mississippi were those that used a stirring or scraping technique. In 1867 $96,000 was appropriated to construct and operate two scrapers on the Upper Mississippi between St. Paul and the mouth of the Illinois. In operation, a dredge equipped with a scraper frame on the bow moved to the upstream end of a crossing, lowered the scraper frame, and then backed slowly downstream, scraping sediment with it into the pool below the crossing. Scrapers were used on the Upper Mississippi throughout the 1868 low-water season and succeeded in increasing depths on short crossings by 8-18 inches. Use of these scraping dredges continued for several years until the decision was made to improve navigation depths by more permanent contraction works (Ockerson, 1898).

Mechanical agitators in a variety of configurations have also been proposed for use on the Mississippi. These devices generally attempt to agitate bottom sediments so that they can be carried away by the current. Although never used extensively on the Mississippi, successful use of agitation dredging was reported recently in the channel leading to the Port of Surabaja in Indonesia where the channel was kept open by means of cylinders dragged along the bed of the channel behind a steamer (Hammond, 1969).

Numerous dredging devices that employ a current deflection principle have also been proposed. An example is the "Machine for Deepening River Channels", patented in 1880, whose purpose was to "deflect the current of a river downward, and thus cause the said current to deepen the channel." A typical deflector proposed for use on the Mississippi in the 1880's is shown in Figure 68a. In this same time frame, Ockerson (1898) reports that the "Kingston" deflector (Figure 68b) was successfully used on the Hooghly River in India where it cut from .5-2 feet through a 300-foot bar in two hours. Current deflectors are, in effect, floating counterparts of the bottom panel, and rely on both helicoidal flow and redirection of the current to produce scour. The Prostov dredging barge is an interesting recent application of this concept. This structure (Figure 68c) consists of a barge to which four vane systems are attached to redirect the flow and produce a spiral flow pattern downstream from the barge. A unit of this type has been tried on the Niger River with satisfactory, though limited results. Experiments on a model of the Prostov dredge were carried out by the French National Hydraulic Laboratory in 1952 with "fairly satisfactory" results. Increasing vane depth produced more extensive erosion, but only over a short distance. By decreasing vane depth an eroding action extending over several times the length of the vane system was obtained. In all cases the width of the eroded channel was less than the width of the structure (NEDECO, 1959).

Mechanical devices have been and still are used for dredging operations on the Mississippi. Grapple or bucket dredges, and ladder dredges equipped with an endless chain of buckets all fall into this category. Dipper dredges, which are essentially barge mounted steam shovels, as well as barge mounted clamshells are also used. For example, the dredging plant of the St. Paul District currently includes the Derrickbarge Hauser, a 4 cubic yard clamshell mounted on a 65-by

45-foot barge. The derrickbarge is used in locations inaccessible to the larger hydraulic dredges and handles approximately 200,000 cubic yards per year at a rate of 2,400 cubic yards a day (Corps of Engineers, St. Paul, 1974).

Experience with contraction work on the Lower Mississippi between 1880 and 1890 convinced the Mississippi River Commission that permanent improvement of the channel would require a long period of time, and that an alternate means must be sought to provide immediate improvement. The Suter-Flad committee was organized in 1891 and tasked to "investigate and report on the most suitable means of affording temporary relief to navigation at low-water stages..." and to recommend "a project for the construction of a dredging boat suitable for such work" (Somervell, 1932).

The Suter-Flad committee examined a wide variety of dredging devices including movable jetties similar in principle to bandals, current deflection panels, and stirring or scraping devices. The rationale for rejecting each of these devices highlights their weaknesses and explains their limited application to date on the Mississippi. Movable jetties were deemed too costly and impractical to place, raise, and move. It was felt that barges with deflectors or leeboards were difficult to anchor without obstructing the channel, and unless kept close to the bottom, deflectors were inefficient. It was observed by the committee that stirring or scraping devices all use the same principle, that is, "to stir up the bottom by some mechanical means, as water jets, harrows, plows, etc., trusting and expecting that the sand thus thrown up from the bottom will be carried off by the current." It was concluded that, although it is a comparatively easy matter to stir up the bottom, the current is inadequate to transport the agitated sediment, except under very favorable conditions. "This has been the invariable experience when the stage of the water has been low enough to make the work a matter of real necessity; and the only success, or partial success, ever attained under these circumstances has been with machines that were calculated to bodily drag away the sand..." (Somervell, 1932).

The Suter-Flad committee concluded that the only alternative that held any chance of success was the hydraulic dredge, and construction of an experimental dredge was recommended. The conclusions and recommendations of this committee initiated the development of hydraulic pipeline dredges of sufficient size to cope with the problems encountered on the Mississippi and established an approach to channel maintenance that persists today.

An experimental dredge was assembled in 1893, and in the fall of 1894 the first attempt to aid navigation was made near Cape Girardeau on a 1600-foot bar where depths ranged from 3 to 4 feet. A 6-foot channel was dredged and this channel remained open throughout the navigation season (Ockerson, 1898). Between 1895 and 1931 eleven hydraulic dredges were built for use on the Mississippi. By 1931 the typical dredge could dig a cut through a sandbar 6 feet deep and 28 feet wide at a rate of 300 feet per hour. It was felt that a satisfactory type of dredge for Mississippi River channel maintenance had been developed (Somervell, 1932).

5.2.2 Hydraulic Pipeline Dredges

The design and construction of hydraulic dredges for use on the Mississippi has been an evolutionary process. Each new design attempted to eliminate the defects of its predecessors and incorporate both technical developments and lessons learned in the field. As a result of this process, two types of hydraulic dredges are currently used for maintenance and improvement dredging in the riverine environment of the Mississippi: the cutterhead dredge and the dustpan dredge. Both types of dredge employ a suction principle, have floating pipelines for discharge, and generally are self-propelled.

The hydraulic cutterhead dredge (Figure 69) is named for its suction device which tapers into a semi-spherical head consisting of a number of large blades. As the head is mechanically rotated, the blades cut into subaqueous material which is then drawn into the suction pipe and discharged by a centrifugal pump through a pipeline. The design of the cutterhead dredge enables it to be employed under a wide variety of conditions encountered in the riverine environment. Consequently, it has been used throughout the Mississippi River system. It can dredge materials ranging from sand to clay to soft rock, and can produce a level bottom in the dredged channel. Thus, it is ideally suited to navigation dredging. Since burial of the cutterhead is of little concern, it can be used to undercut high banks, making it an effective tool for dredging pilot channels for cutoffs and canals.

In operation, the cutterhead dredge generally works downstream against the upstream face of a bar but can be worked upstream. A cut 150 to 300 feet in width must be completed before a through channel is available for navigation. Because of its mode of operation the cutterhead is also known as a swinging dredge. Using winches, cables, and anchors dropped to either side, the dredge swings across a long arc, pivoting on one of two spuds located at the stern (Figure 70). One spud can be used as the operating spud and the other to step the dredge forward as in Figure 70, or both spuds can be used as operating spuds. At the end of a swing the raised spud is dropped, the lowered spud raised, and the dredge swings through the arc again. A cutter normally has a diameter 2 to 2.5 times that of the suction pipe and with every swing removes a rind of sediment a little less than the cutter diameter. When a cut of greater thickness is required several swings are made before the dredge is stepped forward (Figure 70). The cutterhead does not operate efficiently for a depth of cut less than cutterhead diameter.

The dustpan dredge also derives its name from the shape of its suction head which resembles a dustpan or vacuum sweeper (Figure 71). It is best adapted to dredging relatively soft, easily eroded material and has been used extensively on the Lower and Middle Mississippi. With the dustpan dredge, the force exerted on the sediment particles is hydraulic, not mechanical. High velocity water jets near the suction intake agitate the bottom material which is then drawn into the main suction pipe of the dredge and discharged through a pipeline. The dustpan dredge does not perform well in cohesive material, gravel deposits, or against high faces where undercutting could result in burying the suction head.

The dustpan dredge generally is worked upstream. It is held against the downstream face of the bar or crossing by the operation of winches on the dredge which are attached to cables anchored upstream of the bar (Figure 72). The dredge cuts a continuous path the width of the dustpan (up to 32 feet) for the length of the crossing (up to 3500 feet). Greater width is obtained by making additional cuts, starting each time back at the downstream face of the crossing and moving upstream, parallel to the preceding cut. Generally, a narrow ridge, 2 to 5 feet in top width, is left between cuts to be carried away by the current. Unlike the cutterhead dredge, the depth of cut for the dustpan dredge is not restricted to some minimum value. The dustpan can dredge efficiently where only a slight deepening of the channel is desired, thus total dredged quantities can be kept to a minimum. In suitable material it can produce a level cut. These characteristics make it ideally suited to maintaining navigation channels.

The floating discharge pipelines used with both the cutterhead and dustpan dredge are mounted on steel pontoons spaced about 50 feet apart. When current velocity in the river is low, the floating pipeline can be maneuvered and held in position by operation of a baffle or thrust plate at the discharge end of the pipeline. With higher current velocity, anchors placed by auxillary craft are used to hold the pipeline in place.

The dustpan dredge functions most effectively for comparatively short discharge pipelines (500 to 1000 feet). Efficiency drops rapidly for longer lines or for an increase in lift beyond the few feet required by the floating pipeline. Because of the relatively rapid rate of advance of the dustpan dredge, shore pipe cannot be used effectively and dredged material is usually placed in open water. The cutterhead dredge, however, is efficient up to about a 30-foot head with 2000 feet of pipeline. Its mode of operation implies a slow rate of advance along the cut, making the cutterhead dredge well suited for spoiling overbank through shore pipe. Thus, the cutterhead can be used effectively for placing hydraulic fill. Booster pumps can be used to maintain efficiency with longer pipelines.

A cutterhead dredge with a 22-inch intake and 20-inch discharge pipe, cutting a 250-foot width, can advance about 35 feet per hour against a 3-to 4-foot face. In contrast, a dustpan dredge with 24-inch discharge pipe, cutting a 28-foot width, can advance about 300 feet per hour against a 3-to 4-foot face. During normal channel maintenance operations with 1000 feet of discharge pipe both dredges have capacities ranging from 1000 to 1800 cubic yards per hour. Natural conditions such as cohesiveness of the sediment, dredging depth, and current velocity influence capacity. In addition, dredging capacity for both types of dredge is a function of the experience and skill of operating personnel. At best the dredged slurry will consist of 20 percent solids (by volume). Even an experienced lever-man using vacuum and pressure meters to respond to distrubances in the pipeline and to changes in sediment inflow at the suction mouth, will probably not be able to maintain maximum concentration consistently. As a result, the average solids content of the slurry is usually taken as 15 percent. (Feagin, 1947; NEDECO, 1965; Huston, 1970; Schmidt, 1972).

Both the cutterhead and dustpan dredge are well suited to maintenance operations in navigation channels. The cutterhead, with its ability to handle a wide variety of subaqueous materials and greater flexibility of disposal procedures, is certainly the more versatile of the two. In certain instances, such as dredging a crossing which requires only a slight increase in depth to meet project requirements, the cutterhead dredge may operate less efficiently than the dustpan dredge. Either the cutter works less efficiently, as only part of the head is actually used, or the thickness of the cut is adapted to cutter diameter, resulting in the dredging of greater quantities of material than are actually necessary. Since a dustpan dredge makes a narrow but full cut quite rapidly, it is generally assumed that it gains some assistance from the current in making subsequent cuts. Dredge characteristics, then, dictate the manner in which a dredge cut is developed and influence the efficiency of the dredging operation. Dredge capacity, efficiency, and the range of disposal options available at a given location are also a function of both natural conditions and the physical limitations of a specific dredge plant.

5.2.3 Dredging Operations

Dredging operations on the Mississippi River are closely related to the annual cycle of high and low flow. On the Upper Mississippi, for example, channel condition soundings and surveys are initiated following partial recession of the annual spring high water. Survey crews with sonar sounding equipment determine general channel conditions and identify detailed survey requirements. Following these detailed surveys, dredging requirements for the season are determined.

The annual schedule of the Dredge Thompson, the St. Paul District's 20-inch cutterhead dredge, is representative of the timing of dredging operations on the Upper Mississippi. The Thompson is normally dispatched from its service base at Fountain City, Wisconsin to St. Paul between 20 April and 10 May. Usually, critical dredging is accomplished on the way upstream to St. Paul. The Thompson then works downstream from St. Paul accomplishing previously surveyed maintenance dredging requirements. Between August and October the Thompson is utilized by the Rock Island District for channel maintenance dredging in Pools 11 through 22. Upon completion of the Rock Island District requirements, the Thompson performs channel maintenance clean up and then returns to its service base in November. Dredging beyond this time is generally restricted by weather conditions and associated safety hazards (Corps of Engineers, St. Paul, 1974).

The St. Louis District is responsible for channel maintenance in the Pool 24, 25, and 26 study area and on the Middle Mississippi. Two Corps of Engineers dredges, the Kennedy and St. Genevieve, and the commercial dredge Elco constitute the District's normal dredge plant. The Kennedy is a dustpan dredge, while both the St. Genevieve and the Elco are hydraulic cutterhead dredges (Solomon et al., 1974).

The dredging operation at a specific location is based on the detailed survey data provided by the District office. This data includes size and alignment of the cut, and disposal area locations. The cut is laid out either by setting targets on the riverbank or anchoring buoys in the river. A dustpan crew normally sets targets for the initial cut and locates succeeding cuts by sounding for the ridge left by the previous cut. Cutterhead operations are usually guided by targets or buoys either on the centerline or on the outside limits of the cut. Generally, a second survey is made shortly after dredging at a given location. Dredged quantities are estimated by comparing surveys made before and after dredging.

The policy of overdredging the navigation channel in both width and depth is another aspect of dredging operations which could influence both the stability of the dredged cut and the impact of dredging on the river. In the St. Paul District, for example, navigation channel project dimensions are a minimum 9-foot depth and a 300-foot width;

however, channel dimensions on bends are widened up to a maximum of 550 feet, depending on bendway radius of curvature. To insure a 9-foot channel depth, dredging to 11 feet is authorized. The 11-foot requirement is based on experience which indicates that for depths less than 11 feet the propeller wash from towing vessels can cause rapid shoaling to depths as low as 7 feet. An additional 2 feet of overdepth dredging to 13 feet is normally accomplished to allow a reasonable period between maintenance requirements (Corps of Engineers, St. Paul, 1974).

The Mississippi is not the only river on which overdredging has been practiced. In 1957 the Portland District initiated a program of "advance maintenance dredging" on the Columbia River. The results of this well-documented program provide valuable insights into the advantages and disadvantages of overdredging and so are reviewed briefly here.

The Lower Columbia River navigation channel from its mouth at the Pacific Ocean to the confluence of the Willamette River in the Portland-Vancouver area has an authorized project depth of 35 feet and width of 500 feet. The channel is 98.5 miles long and dredging of approximately 9,800,000 cubic yards of river sediment is required annually through 26 river bars whose total length is about 50 miles. The other 48.5 miles of the channel is self-maintaining to at least project dimensions. The river in its natural state had a controlling depth of 12 feet at St. Helens Bar (mile 86) in 1885. Project dimensions are maintained by permanent river contraction works (permeable dikes) and maintenance dredging (Hyde and Beeman, 1963).

Prior to 1957 it was customary to overdredge the Columbia River navigation channel by 2 feet to a depth of 37 feet. Experience indicated that nearly all of the shoaling on the Columbia took place during the several weeks of spring runoff. Maximum shoaling during an average high flow period was 6 to 8 feet, leaving a controlling depth of approximately 30 feet on some bars. To insure project depth throughout the year and to permit scheduling of dredging operations on a year-around basis, a program of advance maintenance dredging was initiated in 1957. The channel that had been dredged to 37 feet was excavated to a depth of 40-42 feet.

The hydrographs of the 1957 and 1961 spring floods on the Columbia were similar in shape as well as peak discharge and stage. Thus the 1957-1961 period provides an opportunity to evaluate the effects of this program on shoaling. A comparison of shoal area and controlling centerline depth on 26 bars between 1957 and 1961 shows a decrease in average shoal area from 994,000 square feet to 539,000 square feet and an increase in average centerline depth from 32.8 feet to 34.2 feet. The advance maintenance dredging program evidently accomplished its objective of providing greater depths throughout the year. It should be noted, however, that these changes reflect the response not only to overdepth dredging, but also to dike construction and extension in this time period.

An analysis which compared the dollar value benefit to shipping to the cost of obtaining additional depth, indicated that the benefit to cost ratio to maintain a 35-foot depth all year was 1.78:1. The cost analysis included not only the cost of the original overdepth dredging but also an increased annual dredging cost related to the change in cross-sectional area resulting from the overdredging. An increased flow area for the same discharge indicates a decreased velocity and thus, an increase in shoaling. It was found that 5 feet of advance maintenance dredging increased the total river cross section on a typical bar by about 5 percent. For purposes of economic evaluation it was assumed that shoaling would be increased by this same factor (Hyde and Beeman, 1963).

In addition to providing economic benefits for navigation, advance maintenance dredging also significantly increased the efficiency of dredging operations. A hydraulic cutterhead dredge does not work efficiently when the depth of cut approaches cutterhead diameter. To assure an efficient dredging operation, a cutterhead dredge should work against the heaviest possible bank. One means of doing this is to overdredge and then allow the crossings to shoal for several years before dredging is repeated. A bar which normally shoals to 2-3 feet a year could be dredged to 6 feet every other year. The increased shoaling resulting from overdredging is compensated for by the increased efficiency of the dredging. For example the production curve for a typical cutterhead dredge (Figure 73) indicates that an increase in dredging bank height from 2 feet to 6 feet doubles dredge production from 750 cubic yards per hour to 1500 cubic yards per hour. Since dredge cost is essentially constant, regardless of production, this would reduce unit costs by 50 percent (Hyde and Beeman, 1963).

The policy of overdredging to allow a reasonable period between maintenance dredging requirements is supported, then, by the economic benefits to navigation that result from the availability of greater depths for longer periods. In addition, where hydraulic cutterhead dredges are used, as on the Columbia and the Upper Mississippi, overdredging results in increased efficiency for the dredging operation. It is apparent however, that these advantages must be weighed against the influence of overdepth dredging on the stability of the dredged cut. In addition, the impact of an increase in the quantity of dredged material requiring disposal must be considered.

5.3 Stability of Dredged Cuts in Alluvium

5.3.1 Analysis

An extensive literature search uncovered only a few references that deal specifically with the hydraulics of dredged cuts and their stability. Maximoff (1904) makes several recommendations based on 20 years of experience with dredging on rivers in Russia. For maximum stability, Maximoff concluded that the dredged cut should have a bottom gradient which increases in the direction of flow, and in plan view, the cut should have a funnel-shape with decreasing width in the direction of flow. There is no indication that these recommendations were tested or implemented. While detailed shaping of the dredged cut may have been possible with the ladder dredges used in Russia during Maximoff's time, the practicality of this approach with equipment currently used for riverine dredging is questionable.

In a series of articles Chatley (1927, 1929, 1938) considered a number of river engineering problems "for which no solution was given in the available literature." The selection of these problems and their analysis was based on 11 years of experience with regulation of the Whangpoo River and study of the Yangtze River estuary in China. Included is an examination of the efficiency of dredging in alluvial channels. Although Chatley's analysis is based primarily on experience with ladder, dipper, and grab dredges, the approach suggested can be adapted to an analysis of the stability of cuts dredged by cutterhead and dustpan dredges.

Dredging of a channel through a crossing or shoal should be considered successful if the dredged cut meets several criteria. First, the cut should achieve the required navigable depth with a minimum of excavation, and second, the cut should not require re-dredging during the navigation season. Minimizing the amount of dredging to a point compatible with the operational characteristics of the type of dredge involved, reduces the cost of the operation as well as associated disposal problems. Alignment, depth, and width of cut all influence excavation requirements. The alignment of the cut in an alluvial river with distinctly different high-flow and low-flow characteristics presents serious problems. In general, no single alignment will coincide with both the high-stage and low-stage patterns of flow over a crossing. Since the dredged channel is required primarily during the low-flow season, the cut is normally aligned with the low-stage thalweg pattern. In terms of minimizing dredged volume, the justification for overdepth or overwidth dredging which increase initial and recurrent dredging requirements must be examined closely, as it was on the Columbia River.

The second criteria, that the cut should not require re-dredging during the navigation season, is also influenced by alignment, depth, and width of cut. Ideally, a successful dredged cut would not require re-dredging at all; however, the characteristics of dredging which cast it in the role of a temporary solution to the navigation problem also preclude achieving this ideal. A decision to align the dredged cut with the low-stage pattern of the river accepts in general that the cut is mal-aligned for high-stage conditions, and thus, cannot be expected to survive more than a few high-flow cycles. Consequently, a dredged cut should be considered stable if it provides the required depths during a single navigation season. For this study the term "stability" will be applied to a dredged cut with reference to the processes of scour and fill in the cut. An unstable dredged cut is one that tends to fill, while a stable dredged cut is one that is self-maintaining, that is, the cut does not show a tendency to fill and may even scour under favorable circumstances.

The stability of a dredged cut is influenced by its alignment, depth, and width. While the alignment of a cut must be related to the configuration and requirements of a specific site, the effect of depth and width on stability can be analyzed for the general case. A simple but instructive analysis of the effect of depth on dredged cut stability can be based on the Darcy-Weisbach equation in a form applicable to uniform or nearly uniform flow in open channels:

$$f = \frac{8gRS}{V^2} \tag{33}$$

where:

f = Friction factor

$R = \frac{A}{P}$ = hydraulic radius = $\frac{\text{Cross-section Area}}{\text{Wetted Perimeter}}$

$S = \frac{h_f}{L}$ = energy gradient = $\frac{\text{Head Loss}}{\text{Length of Channel}}$

V = average flow velocity

Substituting the definition of shear velocity:

$$v_* = \sqrt{gRs} = \sqrt{\tau_o / \rho}$$

where:

τ_o = bed shear stress

ρ = fluid density

yields:

$$f = \frac{8v_*^2}{V^2} = \frac{8\tau_o}{\rho V^2} \quad (34)$$

Rearranging:

$$\tau_o = \frac{f\rho V^2}{8} \quad (35)$$

Thus, for a given channel roughness and constant fluid properties, bed shear stress is proportional to V^2. Recalling the sediment transport formulas of the Du Boys type (Equation 22), the transport of contact load is proportional to bed shear. Consequently, an increase or decrease in bed shear implies an increase or decrease in transport capacity.

Using the continuity equation, (35) can be rewritten:

$$\tau_o = \frac{f\rho Q^2}{8W^2D^2} \quad (36)$$

For a channel with constant width, roughness, and discharge, the bed shear and thus transport capacity will vary inversely as D^2. This relationship indicates why overdepth dredging at the same width of cut increases the tendency to shoal in the cut. If a channel with normal depth of 9 feet is overdredged to 12 feet, bed shear is reduced by approximately 40 percent, and using a contact load transport equation such as (25), transport is reduced by 54 percent.

For an increase in depth at constant width the Colby method offers an alternate approach to estimating the resulting change in transport capacity. Colby's analysis (Section 3.2.4.2) was guided by Einstein's bed-load function, and his graphical relations (Figure 35) incorporate both field and laboratory data. Accordingly, they are more representative of the physical processes involved than a simple shear stress analysis.

Nordin (1971) replotted Colby's curves into a form that is well adapted to an analysis of the effect of width or depth change on transport capacity. For a given bed material size and water temperature, the curves of Figure 35 are plotted as lines of equal sediment discharge per foot of width, q_s, on a velocity-depth field, as sketched in Figure 74. Lines with a slope of minus one are lines of equal unit water discharge, q. The initial velocity and depth, V_1 and D_1, establish the initial unit sediment discharge, q_{s1}, and unit water discharge, q_1. For an increase in depth to D_2 at constant width, q remains constant. Thus, q_{s2} can be established by following the q_1 line to an intersection with D_2 (Figure 74). For example, assuming a median grain size of bed material of .6 mm and a temperature of 60 degrees F, a V_1 of 3.5 feet per second and D_1 of 9 feet establish q_{s1} at 25 tons per day per foot of width. An increase in depth to 12 feet yields a q_{s2} of 5 tons per day per foot of width. A 30 percent increase in depth at constant width produces an 80 percent decrease in transport capacity, indicating that the simple bed shear analysis is quite conservative.

While a constant width analysis sheds some light on the stability problems associated with overdepth dredging, in the general case the influence of width and depth changing concurrently must be considered. As width and depth change, flow area and wetted perimeter also change. Consequently, the influence of width and depth on flow velocity and thus on bed shear and transport can be established through the ratio of area to wetted perimeter, that is, the hydraulic radius, R.

The hydraulic radius is related to flow velocity through uniform flow relationships such as the Chezy or Manning equations. Here, the Chezy equation is used:

$$V = C(RS)^{\frac{1}{2}} \tag{37}$$

where:

C = the Chezy resistance coefficient

R = hydraulic radius

S = energy gradient

Assuming that slope and resistance are constant, velocity is proportional to $R^{\frac{1}{2}}$, and the impact of dredging on the hydraulic radius of a section can be used as a criteria for stability. If the dredging process adds to the wetted perimeter faster than it increases flow area, R and V decrease, indicating a tendency toward shoaling in the dredged cut. Conversely, if area increases faster than wetted perimeter, R and V increase, indicating possible scour in the cut or at least a tendency toward stability.

Although a dredged cut is roughly trapezoidal in shape with side slopes near the angle of repose of the bed material involved, a rectangular cut has been selected for simplicity. Analysis of a trapezoidal section would yield similar results. Figure 75 is a definition sketch for a rectangular dredged cut in depth of water, D. The depth of cut is Δ and the width of cut is shown in increments of Δ as $n\Delta$, where n = 1, 2, 3...etc. Applying the Chezy equation (37) to the undisturbed condition:

$$V_o = C(R_o S)^{\frac{1}{2}} = C[\frac{(n\Delta)(D)}{(n\Delta)} S]^{\frac{1}{2}} \tag{38}$$

After dredging:

$$V_1 = C(R_1 S)^{\frac{1}{2}} = C[\frac{(n\Delta)(\Delta+D)}{(n\Delta+2\Delta)} S]^{\frac{1}{2}} \tag{39}$$

For no change in velocity, $R_1 = R_o$ or:

$$\frac{D}{n\Delta} = \frac{(\Delta+D)}{(n\Delta+2\Delta)} \tag{40}$$

and:

$$n\Delta = 2D \tag{41}$$

Thus, the velocity is unchanged if the width of the cut (expressed in increments of Δ) is equal to twice the undisturbed water depth. For $n\Delta > 2D$ flow area increases faster than wetted perimeter and $R_1 > R_o$. For $n\Delta < 2D$ wetted perimeter increases faster than flow area and $R_1 < R_o$.

For a given depth of flow, D, values of R_1 can be computed for various combinations of n and Δ. For an undisturbed depth of 16 feet, Table 13 shows R_1 values for n and Δ combinations selected to reflect the operational capabilities of hydraulic cutterhead and dustpan dredges. Since $R_o = D$, the stepped line in Table 13 represents the equilibrium condition of $R_1 = R_o = 16$, or $n\Delta = 32$. It should be noted that since:

$$R_1 = \frac{n(D+\Delta)}{n+2} \tag{42}$$

as $n \to \infty$, $R_1 \to (D+\Delta)$. Thus, there is an upper limit to the increase of R_1 for each depth of cut, Δ.

A more general formulation of this data is shown in Figure 76 where values of R_1 and n are plotted with Δ as a third variable for a rectangular dredged cut and an undisturbed depth of 16 feet. Combinations of n and Δ which plot above the $R_1 = R_o$ line indicate that a cut with that depth, Δ, and a width of $n\Delta$ should tend toward stability. Values of n and Δ which plot below the line indicate instability or a tendency toward shoaling. For example, for a 2 foot depth of cut a width of 60 feet (n = 30) would be stable, while a width of 20 feet (n = 10) would be unstable.

Cutterhead and dustpan dredges develop a dredged cut in a distinctly different manner. An analysis based on the hydraulic radius of the cut indicates that each mode of operation has certain advantages and disadvantages. The single, wide cut of the cutterhead dredge provides the opportunity to immediately maximize the hydraulic radius of the cut for a given depth. For example, at a depth of cut of 5 feet and a width ($n\Delta$) of 300 feet Figure 76 shows a hydraulic radius of 20.4 feet. This represents an increase of 28 percent over the equilibrium hydraulic radius of 16 feet, which in turn could produce a local increase in velocity of 13 percent. The dustpan dredge, however, is restricted to a width of cut of about 30 feet, and is just able to maintain an equilibrium hydraulic radius of 16 feet at a depth of cut of 3 feet.

The speed with which the cut is developed may also effect its stability. Although a single narrow cut of the dustpan dredge is only marginally stable, each cut is developed quite rapidly. A through channel is produced with the first cut, and some assistance is normally gained from the current in removing the narrow step left between each successive cut. The rapid development of a through channel whose width and hydraulic radius are progressively increased could compensate for the marginal stability of the individual narrow cuts. In terms of stability of the cut produced there is apparently no clear advantage for either the cutterhead or dustpan dredge.

Using the Chezy equation in a discharge form:

$$Q = C(S)^{\frac{1}{2}} \Sigma WD^{3/2} \tag{43}$$

NEDECO (1959) analyzed the potential impact of a dredged cut on a typical crossing of the Niger River. The proposed cut (Figure 77) was 2000 feet long by 300 feet wide with a 3-4 foot depth of cut. Cross-section A-A in Figure 77 was drawn at right angles to the flow lines and over the crest of the crossing at a stage of low river level plus 3.33 feet. The conclusions resulting from this analysis of the impact of a dredged cut on the flow over a crossing include:

1. The factor $\Sigma WD^{3/2}$ increases, as a result of the dredged cut, by 5 percent, which would produce a decrease in slope over the crossing of 10 percent (assuming constant Q and C).

2. The cut would create some concentration of flow lines on the upstream portion of the crossing. An extra 3500 cfs would be drawn through the cut, however, this represents only 3 percent of the total discharge at the selected stage.

3. The overall flow pattern over the crossing alters only slightly, as the cross section of the cut is quite small when compared to the total cross section of the river.

The change in water surface profile over a pool and crossing sequence is shown in Figure 16 for high and low stage. At low stage a relatively steep water surface gradient will exist over the crossing, contributing to scour on the crossing under natural conditions. Since dredging produces only a small decrease in slope over the crossing, NEDECO concludes that at low stage a higher slope and a higher sand transporting capacity can still be expected over the crossing than over the pool immediately upstream. Some scour can be expected on the dredged crossing, provided the depth of cut is not too great. This limiting depth is generally taken by NEDECO as no greater than the hydraulic depth of the section (flow area/surface width) below the low river level.

Combining a possible 28 percent increase in hydraulic radius in the cut from optimizing section shape with a representative 10 percent decrease in slope over the crossing as a result of dredging, the Chezy equation (37) indicates a net 7 percent increase in velocity. Using equation (35) this would result in a 15 percent increase in bed shear in the cut, and from equation (25) a 24 percent increase in contact-load transport capacity. There is some indication, then, that under optimum conditions a dredged cut can be stable. The effect of alignment on stability and the influence of unsteady flow resulting from a normal seasonal hydrograph are examined in subsequent sections.

NEDECO (1959) estimated the pattern of scour and fill in the typical dredged cut (Figure 78) based on contact load measurements over a natural crossing. These measurements indicated a maximum decrease in sand transport capacity at the point where the flow enters the pool below the crossing. Coupled with the results of the Chezy equation analysis these measurements point to a pattern of scour in the upstream portion of the cut and deposition on the downstream rim of the crossing. The trend line of Figure 78 represents the expected bottom profile about one month after dredging. The tendency to build a shallow bar of sand just below the crossing would be counteracted by increased transport as the water depth decreased. The result would be the growth of a fan-like deposit into the pool below the crossing.

The following conclusions result from the NEDECO (1959) analysis:

1. The hydraulic depth on the crossing at the average low water discharge is a limiting factor. If dredging is limited to only this depth, reshoaling does not seem likely.

2. The greatest deposit of sand can be expected at the downstream end of the dredged channel and in the pool below the crossing. While deposits on the downstream rim of the crossing may require maintenance dredging, deposition in the pool will not usually effect navigation.

3. Natural repositioning of the channel over the crossing may cause shoaling and require maintenance dredging. Experience in choosing the right alignment for the cut will reduce the chances of this happening.

The complexity of alluvial channel flow (Equation 1) makes it improbable that a simplified analysis based on the few variables in a uniform flow equation can do more than indicate a tendency toward stability or instability in a dredged cut. Additional information relative to the stability of dredged cuts in alluvium can be obtained from test dredging in the field, hydraulic model studies, and correlation of dredging requirements with certain hydraulic parameters. These areas are addressed in the following sections.

5.3.2 Test Dredging

When the potential benefits of a well-documented test dredging program are considered, there are surprisingly few cases in which test dredging has been conducted and reported on in sufficient detail to be of value. One of the earliest cases, reported by Ockerson (1898), provides a striking example of the mobility of the bed of an alluvial river and the fate of a dredged cut. During the low-water season of 1896 the Lower Point Pleasant Bar, 79.5 miles below Cairo, was one of eight crossings which obstructed navigation on the Lower Mississippi between Cairo and Memphis. One day before dredging (Figure 79a) there was barely a 7-foot channel between the upper and lower pools. The dashed line indicates the location and orientation of the proposed 200-foot by 1700-foot dredged channel between the two pools. Figure 79b, two days after completion of a 3-foot dredged cut, shows the two pools connected by a 9-foot channel with 11 feet of water for the greater part of its length. Seventeen days after dredging (Figure 80a) an 11-foot channel exists from pool to pool, with nearly 13 feet of water most of the way. The dredged cut experienced a significant amount of natural scour during this two-week period. Figure 80b, 33 days after dredging, shows a striking change. The channel was now almost 500 feet downstream from its location of 16 days before, and the site of the dredged channel now had only 4 feet of water. A channel with 9-foot depths remained between the pools, but displaced downstream. This change occurred during a rapid rise in stage at the end of the 1896 low-water season.

In response to a request from the Chief of Engineers and the President of the Mississippi River Commission in 1931, the Memphis

District initiated a program to document the results of normal maintenance dredging activities and perform some experimental dredging on the Lower Mississippi. The subsequent report from the Memphis District (Somervell, 1932) included a summary of dredging activities in the vicinity of Island 35, a particularly troublesome reach of the river. Since 1920 the main channel, a long meander loop to the right of Island 35, had been filling rapidly, and the chute channel to the left of the island had widened and deepened. In 1927 attempts were initiated to maintain a navigation channel through the chute. The dredged cut required the removal of some 660,000 cubic yards of material to secure a clear channel 300 feet wide and 20 feet deep (almost double project depth). A subsequent rise in stage of 12 feet followed by a sharp fall placed 461,200 cubic yards in the cut itself and in the 1.18 square miles immediately contiguous to the cut a fill of 4,629,000 cubic yards occurred. Experimental dredging in 1931 in this vicinity, together with the work considered as normal maintenance, amounted to a total of 2,186,831 cubic yards. It was necessary to redredge one area in this vicinity ten times, one five times and one four times, during the 1931 season. The 1931 hydrograph shows that the low water season was characterized by frequent fluctuations. In the light of previous analysis the extreme overdepth dredging may have contributed to the apparent instability of the initial dredged cut. The sensitivity of the dredged cut to rise and fall in stage is also quite apparent and is evaluated in more detail in a subsequent section.

A more recent experimental dredging project resulted from NEDECO'S (1973) Rio Magdalena and Canal del Dique Survey in Columbia. This project was carried out jointly by NEDECO and the Columbian government's

Associación Nacional de Navieros (ADENAVI) to obtain as much information as possible about the stability and behavior of a cut dredged prior to a low-water period. Although dredging on 3 crossings was planned, the program of before, during, and after dredging activities was carried to completion at only one location, the Sogamoso crossing, which was selected as a typical example of a crossing on the Rio Magdalena. The planned cut at Sogamoso was to be 1600 feet long, 200 feet wide, and to insure a depth of 7.5 feet below low river level. Using a 20 inch dustpan dredge with a 33-foot dustpan width, 3 parallel cuts were required to produce the desired channel width (Figure 81).

The problems encountered in the execution of this program were numerous and provide some explanation of the paucity of good experimental field data on dredging in the literature. The test dredging on the Magdalena was scheduled for June/July 1973, the normal pre-low water period on the river. Unfortunately, water levels in June/July 1973 were 2 meters higher than usual, and the water level drop during and after dredging was negligible. As a result the expected scour on falling stage (retarded scour--Section 3.2.2.3) did not occur. Equipment difficulties were also encountered, particularly with the discharge pipeline. As a result of frequent breaking of the pipeline, only 90 meters of discharge pipe were available. The point of discharge of the dredged material was about 45 meters from the dredged cut so that some of the sediment was dredged twice. It was also observed that a portion of the dredged material washed back into the channel downstream of the dredged cut. In addition, uncertainty as to the reference station used to reduce the soundings make the before and after dredging data somewhat suspect.

As a result of this program it was concluded that 3-4 cuts with a dustpan dredge were sufficient to produce a 200 foot wide channel. Some assistance was obtained from the current in removing material from the cut, however, overdepth dredging was required to assure desired channel depths. Observations several weeks after dredging indicated that the middle part of the cut sedimented rapidly as a result of crosscurrents, but the head of the cut maintained its depth although displaced slightly downstream.

5.3.3 Model Studies

Because of the complexity of alluvial channel flow, analytical solutions to river engineering problems are often difficult to obtain. This is particularly true when the movement of sediment is a significant part of the problem, as it is in the analysis of the stability of dredged cuts. Mobile-bed hydraulic models are frequently used to provide solutions to these complex alluvial channel problems. Unfortunately, the application of hydraulic modeling techniques to the problem of dredged cut stability has been quite limited. One of the few instances in which the dredged cut and its stability was more than a peripheral aspect of the problem being investigated was in a model study of shoaling problems in the Manchester Islands reach of the Ohio River. Because this study involved selecting an optimum orientation for a dredged cut in a typical divided reach as well as investigating the fate of dredged material disposed at various locations, a review of the study and an analysis of resulting data is pertinent to this and subsequent sections.

The Manchester Islands at mile 396 below Pittsburgh, Pennsylvania divide the Ohio River into three channels (Figure 82), the Kentucky channel, the Ohio channel and the middle channel between the islands. The navigation channel at the time of the model study followed the Ohio bank above and below the islands, and the Kentucky bank past the islands from mile 397.2 to mile 394.5 with project dimensions of 9 feet in depth and 500 feet in width. Although the navigation channel was the widest of the three, it was also the longest since it followed the concave side of the bend. The other two channels were straighter, but too shallow or too narrow for navigation at normal pool level. The Manchester Islands are located in the pool of Lock and Dam 33, a navigation lock and dam similar in structural configuration and mode of operation to the locks and dams of the Upper Mississippi (Figure 82).

During high-water periods the Kentucky channel shoaled to the point that periodic dredging was required to maintain navigation depth and width. Dredging operations during June 1935 removed 235,000 cubic yards of sand and gravel from this channel. A survey made in July 1936 indicated that about 530,000 cubic yards of additional material would have to be removed to restore the 1935 after-dredging conditions. Since an analytical solution to the shoaling problem was "practically impossible," the decision was made to make use of a small-scale model to study the effectiveness of various proposed improvements. Specifically, the model study sought to determine the optimum location for a dredged navigation channel past the islands, and the most effective plan for maintaining that channel in the location selected. Methods of improving dredging procedure, particularly with regard to dredged material disposal, were also studied. To support the study of the fate of dredged material disposed at various locations, an analysis of current velocity and direction at low, intermediate and high stages was also undertaken (Corps of Engineers, Waterways Experiment Station, 1941).

To insure duplication of flow conditions above and below the problem area, the Manchester Islands model was constructed to reproduce the channel of the Ohio River between miles 392 and 400 (Figure 82). Model limits included all of the overbank area to an elevation above the high water of 1937. Since the essence of the problem under investigation was scouring and shoaling in the channel bed, the proper simulation of bed-load movement was critical. The channel bed below normal pool level was molded of crushed coal, providing a bed material free to move in simulation of bed-load movement in the prototype. The bank and overbank areas as well as locations in the channel where rock or gravel were known to exist were molded in concrete. The linear scale ratios (model to prototype) were 1 to 300 in the horizontal, and 1 to 80 in the vertical dimension.

Initially two improvement plans were to be tested (Plans A and B). During the course of the study a third plan evolved (Plan C) and in addition, numerous variations of these three basic schemes were tested. Plan A (Figure 84) involved the dredging of the existing Kentucky channel to project dimensions and the closing of the middle channel by means of a dike of dredged material. Plan B (Figure 85) involved the closing of the existing channel on the Kentucky side by means of a dike of dredged material at the upper end of Island no. 1, and the dredging of a new navigation channel between the two islands. Plan C (Figure 86) envisioned dredging the navigation channel through the Ohio channel and partially closing the Kentucky channel with a dike of dredged material.

In each case a test was begun with the model bed molded to the configuration of the July 1936 river survey. The regulating works and dredged cut were installed in the model after the bed was molded but prior to the beginning of model operation. The test of any given plan was accomplished by subjecting that configuration to successive applications of a hydrograph approximating that recorded at Lock and Dam 33 between 1 July 1935 and 1 July 1936. Each test was continued until repeated application of this hydrograph produced a stabilized model bed.

Prior to testing Plans A, B, and C, a verification test was made to calibrate the model. During calibration, hydraulic parameters of the model were adjusted until model conditions closely reproduced those of the prototype. Following verification, base test runs were made to establish prototype conditions prior to modification. Results of verification and base test runs provide considerable insight into the existing problems of maintaining the Kentucky channel for navigation. Channel conditions as of August 1935, shortly after dredging in the prototype (May--July 1935), were closely reproduced in the model (Figure 83). As with the prototype, significant shoaling occurred in the model at the upper end of the dredged cut. During the 1935 dredging season two primary disposal sites were used for dredged material: the head of Island No. 1 and the middle channel between the two islands (Figure 83). In both model and prototype a large bar was formed at the foot of Island No. 1. The model revealed that this bar was formed almost entirely of material carried downstream from the disposal area at the head of Island No. 1. Below Island No. 1 the stability of the dredged cut was affected by material disposed in the middle channel. During high stages material from this disposal site was carried downstream along the bank of Island No. 2 and produced shoaling in the dredged channel below Island No. 1. This pattern is reminiscent of shoaling patterns in the Long Island reach of the Niger River investigated by NEDECO (Section 3.2.3.3).

Under existing conditions in the prototype, the Kentucky channel carried about 52 percent of the flow during low stages, and about 49 percent during high stages (Table 14--Base Test). During high flow, material was scoured from the point bar in the next bend upstream (Figure 83--Pennyweight Bar) and deposited in the reach above the Manchester Islands, causing shoaling in the head of the dredged cut. An examination of current velocity and direction measurements in the model at high stage indicated that the thread of maximum velocity cut across the head of the dredged cut and then divided around Island No. 1, following the south bank of Island No. 1 through the Kentucky channel and the south bank of Island No. 2 through the middle channel. In exiting from the middle channel, the path of high velocity flow again cut across the alignment of the dredged cut below Island No. 1. At low flow the main current in the reach followed the Kentucky channel, and its direction closely coincided with the alignment of the dredged cut. Here again the impracticality of aligning a dredged cut with both high and low-flow channel patterns is apparent (Section 3.2.2.3). The consequences are also apparent. The unstable conditions in the dredged channel above and below Island No. 1 can be related to strong high-stage crosscurrents that impinge on the cut at each location. Both locations are also downstream from a high-stage sediment source area. Pennyweight bar contributes material that deposits in the head of the dredged cut, and dredged material disposal sites above and between the two islands

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contribute material that form the bar below Island No. 1, impacting the dredged channel there.

Under Plan A the Kentucky channel was dredged to 12 feet below normal pool level with an average width of 500 feet. The middle channel was closed by a dike constructed of dredged material to an elevation of eight feet above normal pool (Figure 84). Dredged material not required to construct the dike was placed along the south bank of Island No. 2. As with the verification and base runs, considerable shoaling occurred in the crossing and head of the dredged cut above the islands. Material forming this shoal came mainly from Pennyweight Bar upstream. This material was deposited on the crossing at about mile 394.5 during high stages and was moved into the upper end of the dredged cut during low stages. By the end of the test (five repetitions of the hydrograph) the dredged cut in this shoal area had been obliterated.

Closure of the middle channel under Plan A resulted in significant redistribution of the flow through the divided reach. At stages lower than five feet above normal pool, the dike between the islands prevented any flow through the middle channel (Table 14). At low stages most of the discharge formerly carried by the middle channel swept across the 1935 dredged material disposal site at the head of Island No. 1 and into the Kentucky channel. Material scoured from this site was swept downstream and deposited on a bar at the foot of Island No. 2. The increase in discharge through the Kentucky channel caused some erosion on the bar downstream from Island No. 1 with the material being deposited just downstream and toward Island No. 2. The disposal area along the south bank of Island No. 2 remained quite stable, however at high stage some material was eroded from this site and carried downstream.

Although Plan A did not eliminate shoaling on the crossing above Island No. 1 or in the head of the dredged cut, shoaling below Island No. 1 was greatly reduced. The redistribution of discharge (Table 14) and modification of current velocity (Table 15) in the reconfigured channel provides an explanation of the increased stability of the lower end of the dredged cut. As a result of closure of the middle channel, discharge increased, on the average, by 16 percent through the Kentucky channel and velocity increased by 12 percent. The corresponding increase in transport capacity was apparently sufficient to produce stability in the lower end of the dredged cut and, in fact, induce scour on the bar below Island No. 1. In the middle channel, discharge decreased by 56 percent and velocity by 60 percent. As a result, dredged material placed along Island No. 2 was not moved into the lower end of the dredged cut as it had been during the base tests.

Plan B involved the construction of a dike across the Kentucky channel and dredging a 12-foot deep by 500-foot wide navigation channel between the islands (Figure 85). The dredging required the removal of the equivalent of about 936,000 cubic yards of material (referred to prototype dimensions). Dredged material not required for construction of the dike was placed along the south bank of Island No. 2 between miles 396 and 396.5. The dredged cut, dike, and general configuration of the model are shown in Figure 87.

Again, significant filling of the head of the dredged channel occurred as high-stage flows moved sediments off of Pennyweight Bar. Velocities of subsequent low-stage flows were not sufficient to scour this material. In addition strong crosscurrents were noted over the dredged cut above Island No. 1 as the dike deflected flow away from the Kentucky channel toward the middle channel, and these contributed to further shoaling. Discharge through the middle channel increased, on the average, by 124 percent while velocities increased by 37 percent. This increase in discharge and velocity was sufficient to maintain a channel of project depth and width below Island No. 1 and along the Kentucky bank downstream. Intense scour was noted on the disposal area along Island No. 2, however, the material was deposited downstream from Island No. 2 and out of the navigation channel.

The configuration for Plan C included a 12-foot deep dredged cut through the Ohio channel, with a minimum width of 350 feet at mile 396.3. The cut required removal of 955,000 cubic yards of material. A portion of this material was used to construct a dike from the Kentucky bank to the old disposal site above Island No. 1 (Figure 86). The remainder was placed along the south bank of Island No. 2. With this alignment, a continuous channel of project depth was maintained along the Ohio bank and into the dredged cut. Material carried downstream from Pennyweight Bar by high-stage flows deposited on and above the 1935 dredged material disposal site at the head of Island No. 1. During low-stage flows some of this material was moved across the disposal area toward the Ohio channel, but high velocities in the entrance to the middle channel intercepted this material and carried it through that channel. As a result of diking the Kentucky channel, discharge and velocity through the middle channel increased by 23 percent and 22 percent, respectively. Discharge through the Ohio channel increased, on the average, by 53 percent, while velocity increased by 19 percent.

Peak velocities as high as 9.9 feet per second in the middle channel and 7.3 feet per second were recorded (Table 15). These velocities produced severe scour in both channels and resulted in large quantities of material being deposited in the lower end of the navigation channel and below Island No. 2. With this configuration dredging of an additional crossing below mile 397 toward the Kentucky bank would be necessary.

Neither the basic plans described above nor variations based on the concept of increasing the proportion of the discharge passing through a dredged navigation channel by closing one of the other channels produced an entirely satisfactory result. Two additional plans based on natural tendencies of the river, as revealed by verification and base tests, appeared to provide reasonably satisfactory solutions. These plans involved dredging the existing navigation channel and installing training dikes between the head of Island No. 1 and the Ohio bank to increase the percentage of flow through the Kentucky channel at all stages. These training dikes also served to confine the flow at the head of the navigation channel in the section above the islands where deposition tends to occur. These plans also had their disadvantages, including strong currents and resulting scour near the training dikes and the creation of velocities in the channel of sufficient magnitude to constitute a hazard to navigation. Consequently, it was concluded that the most practical immediate solution of the shoaling problem was improvement in the dredging procedure, particularly with regard to locating dredged material disposal sites to insure that the dredged material would not find its way back into the navigation channel.

This model study highlights several significant aspects of dredged cut stability. The criticality of alignment is apparent. Equally apparent is the difficulty of aligning the cut to conform to both high-stage and low-stage flow patterns. While the dredged cuts for Plans A and B were oriented properly for low-stage flows, high-stage flows produced crosscurrents that in both cases resulted in shoaling at the head of the cut. Under Plan C both low and high-stage flows were forced into the navigation channel and shoaling at the head of the channel did not occur. Equally important is the location of sediment source areas such as an upstream point bar or dredged material disposal sites. When these source areas above or adjacent to a dredged cut are subjected to high velocity, high-stage flows, serious shoaling in the cut can result. The problem is particularly serious when high velocity, sediment laden currents cut across the alignment of the dredged cut as with Plan A above and below Island No. 1, and with Plan B above Island No. 1. Those portions of the dredged cut that are properly aligned for both high and low-stage flows, and where velocities are sufficient to induce some scour, appear to be quite stable. However, extremely high velocities in a dredged channel can place so much sediment in motion that serious deposition problems result at the downstream end of the cut, as with Plan C. It is significant that in this case continued dredging apparently offered the most practical solution to the shoaling problem in the Manchester Islands reach. Although the more permanent approach of installing training dikes solved the shoaling problem, secondary effects made this solution unacceptable. Here dredging represents a compromise by temporarily solving the shoaling problem and avoiding undesirable secondary effects of more permanent solutions.

The Manchester Islands model also provided a unique opportunity to investigate the fate of dredged material disposed at various locations in a divided reach. As this subject is more appropriately addressed in a subsequent section, consideration of this aspect of the model study is deferred until then.

While only a peripheral aspect of the problem under investigation, the stability of a dredged cut is alluded to in two additional model studies by the Waterways Experiment Station. Both studies relate to the development of a navigation channel on the Arkansas River. In the first, a movable-bed model of a 10-mile reach of the Arkansas River was used to study general problems involved in the development and maintenance of a navigable channel. With regard to dredging, it was concluded that a dredged cut without appreciable reduction in the sediment load or the addition of regulatory works would only temporarily improve channel conditions in a given reach (Corps of Engineers, Waterways Experiment Station, 1962). This conclusion supports previous discussion of the role of dredging in creating navigation channels.

In the second study, a movable-bed model of 11 miles of the Arkansas River bracketing Lock and Dam 8 was constructed. Among the purposes of the study was the determination of the relative effectiveness of various sizes of dredged channels in the section downstream from the dam. Here, it was envisioned that dredging would be used to accelerate the development of the navigation channel below the dam, since the natural process of scour would require too long to develop necessary channel depth and width. It was concluded that increasing the size of the initial dredged cut in the reach below the dam would tend to reduce the amount of maintenance dredging required during the first two years, but would tend to increase the total amount of dredging required for the initial cut and maintenance dredging (Franco and McKellar, 1973). This conclusion supports earlier discussion of the consequences of overdredging.

5.3.4 The Influence of Stage

The influence of stage on the transport of water and sediment through the crossing and pool sequence of a meandering thalweg river is reviewed in Chapter 3 (Section 3.2.2.3). The effects of changing patterns of flow at high and low stage on the stability of a dredged cut are clearly evident in the Manchester Islands model study. Further, the influence of stage is apparent in the two experimental dredging programs on the Mississippi previously outlined. Instabilities in experimental dredge cuts at both the Lower Point Pleasant Bar (Ockerson, 1898) and Island 35 (Somervell, 1932) were related to either a rapid rise and fall or a series of fluctuations in stage (Section 5.3.2). The sensitivity of a dredged cut to changing stage is sufficiently important to warrant additional consideration.

As part of the program initiated by the Memphis District to document the results of maintenance and experimental dredging activities on the Lower Mississippi, data was taken during the 1930 and 1931 dredging seasons to relate river stage and dredging requirements (Somervell, 1932). Although it was generally accepted at the time that crossings or bars are built up on falling river stages which follow high and intermediate flows and are scoured at lower stages, data was not available to support this contention. To establish the relationship of stage, depth over crossings, and dredging requirements, detailed measurements of depth on 34 crossings between Hickmann, Kentucky and

Memphis, Tennessee (191 River Miles) were taken during the 1930 and 1931 dredging seasons. These measurements for 1930, averaged over 10 day periods, are compared with average stage at Memphis and related to dredging requirements in Table 16. Also shown are the trends of the stage (rising, falling, or stationary) and the number of dredges mobilized in the District to maintain the navigation channel during the period. This information is summarized graphically in Figure 88.

The peak of the hydrograph in Figure 88 and the least available average depth over the crossings both occurred during the 21-30 June period. This peak was followed by rapidly falling stages until the end of August and then relatively stationary water levels until November. The combined action of the river and the dredging necessary to maintain a channel of project dimensions during this period lowered the average elevation on the crossings by 6.2 feet. The rapid increase in dredging required to maintain the navigation channel between 20 July and 10 August as stages fell provides a strong indicator of the detrimental effect of rapidly falling stage on dredged cut stability. The 1.5 foot rise and fall of stage in late September was followed by a period of slightly fluctuating stages and resulted in significantly increased dredging requirements during November. A 2.7 foot rise in stage during the November-December period was accompanied by a 2.4 foot decrease in average depth on the crossings.

Data for the 1931 dredging season exhibits the same trends. A rise in the river in September 1931 was followed by a period of strong fluctuations in October and November. Stage decreased by 3.6 feet in late October and increased by 9.2 feet in late November. These fluctuating stages were accompanied by a greatly increased dredging requirement. During the 40 day period, 21 October to 30 November 1931, almost 1,475,000 cubic yards of dredging was required to maintain the navigation channel. During the same period in 1930, with slowly rising stages, 870,000 cubic yards of dredging was required.

An excellent indication of the effect of periods of prolonged high and low water can be obtained from soundings on 27 crossings made by the Memphis District at the beginning of the 1929 and 1931 low water seasons. During 1928, 1930, and 1931 the Mississippi River at Memphis did not exceed flood stage (35 feet), however, during 1929, the river exceeded flood stage at Memphis for 88 days. Thus, 1929 represents a relatively long period of high water while 1930-31 represents a prolonged low-water period. The high-water period of 1929 significantly reduced average depths over the 27 crossings sounded, and was followed by such a rapid fall in stage that the amount of retarded scour was well below normal. As a result, the average depth on a crossing was 5.29 feet below Memphis mean low water. Following the prolonged low-water period of 1930-31, the average depth on a crossing was 12.78 below the Memphis mean low water, an increase in 7.5 feet in average depth on the 27 crossings. This indicates the pronounced influence of stage on depth over a crossing, and consequently on dredging requirements.

More recently, the 1973 flood on the Mississippi River provided graphic evidence of the influence of stage on dredging requirements. This flood produced a record high stage of 43.3 feet at St. Louis (Figure 65). An important and unique aspect of the flood was its duration. Above normal stages on the Mississippi began in November 1972 and did not subside until July 1973, a nine month period. In the

St. Louis District 380 miles of navigable waterways are maintained on the Mississippi, Missouri, and Illinois Rivers combined. Normally, about 30 crossings on these three rivers require maintenance dredging. After the 1973 flood, however, maintenance dredging was required on 32 crossings on the Mississippi, 19 crossings on the Missouri, and 9 crossings on the Illinois, a total of 60 crossings. Not only did the number of dredging trouble spots on these three rivers double, but dredging volumes also doubled to an estimated 14,000,000 cubic yards as a result of the 1973 flood (Hakenjos, 1974).

River engineers in the Soviet Union used the relation between stage and depth over crossings to plan navigation works as early as 1892 (Kondrat'ev et al., 1962). It was found that navigation conditions varied considerably during the navigation season, and from year to year, depending on the character of the spring high water. In an investigation of crossings on the Don River in 1940, Polyakov showed that in years with "high and intensive water level rises" during the high-water period, the crests of the shallows can grow as much as two or three meters, while the pools show corresponding high erosion.

In 1940 Kustov developed a method for predicting dredging requirements based on the relationship between depth on the crossing and stage at the nearest gaging station. As there is considerable scatter in this relationship (Figure 89), boundary curves are drawn which indicate the limits of high and low depth on the crossing based on observations over a period of years. The upper boundary represents the limit of the least available depth for a given stage at that crossing, and the lower boundary indicates the maximum available depth. On the same graph the curve of minimum guaranteed depth for that reach of river as established by the Navigation Authority is also plotted (Figure 89). The intersection of the minimum guaranteed depth curve for the reach and the curve of least available depth on the crossing establish the stage at which dredging on that crossing must be anticipated. For the crossing shown in Figure 89 this stage would be 1.2 meters on the reference gaging station. The depth of dredged cut required is also determined from this graph as the difference in depth between the upper and lower boundaries at the stage requiring dredging, in this case 3.0-2.4 or .6 meters. It is of interest that this approach insures that dredging is done only to the maximum depth that the river can maintain naturally on the crossing. This is in consonance with NEDECO's recommendation of dredging only to the low-water hydraulic depth on a crossing to minimize reshoaling (Section 5.3.1).

5.4 Dredging as a Morphologic Agent

Although dredging operations in the riverine environment are generally maintenance oriented, there are certain situations in which the dredge can be viewed as a development tool. As such, the dredge provides the river engineer with a means of rapidly altering channel configuration and accelerating morphologic processes in support of river development programs. In this respect dredging constitutes a morphologic agent responsive to engineering requirements. This is the context in which dredging is viewed in this section.

5.4.1 Development Dredging

An excellent example of the use of dredging to accelerate morphologic processes as an integral part of river system design is provided by the Arkansas River project. This project envisioned the development of the Arkansas River for navigation, flood control, hydroelectric power generation, and other uses by means of upstream storage reservoirs and a series of navigation locks and dams, similar to those on the Upper Mississippi. Estimates indicated that the trapping of sediment in the upstream storage reservoirs and in the larger pools of the main stem navigation system would reduce the existing 100,000,000 ton per year sediment load by about 90 percent, and would also induce extensive degradation of the streambed downstream from the navigation dams. Project design included taking advantage of this degradation by increasing the spacing and reducing the number of navigation dams from that required to match the natural river profile. The navigation channel just downstream from each lock and dam was to be developed initially by dredging and contraction work to accelerate the anticipated natural degradation, and was designed to conform to the modified regime conditions of the channel (Madden, 1964).

The model study of Lock and Dam 8 on the Arkansas River (Franco and McKellar, 1973) referred to previously, was intended, in part, to determine the optimum width and depth of a dredged cut to develop the desired channel below a lock and dam. Based on previous experience with similar systems, it was anticipated that natural degradation would progress slowly in the reach downstream from a lock and dam, and that this degradation would be retarded and eventually arrested by the formation of an armoring layer of gravel on the bed surface (see for example: Livesey, 1963; Halmark and Smith, 1965; or Komura and Simons, 1967). Dredging was designed to provide a navigable channel at the time the locks and dams were completed in each section of the river, and to inhibit the formation of a gravel armor layer prior to the development of the project channel. Armoring that occurred after dredging was considered beneficial in stabilizing the dredged channel.

Dredging quantities were estimated as the volume of material equivalent to that between the cross section of the bed at the time the upstream lock and dam was put in operation and the design equilibrium bed cross section for modified sediment transport. A dredge can operate more efficiently when excavating a deeper cut than would normally conform to the relatively shallow depth of material above the design equilibrium cross section. Consequently, it was proposed that the required volume be removed in a narrow, deep cut, permitting the river to remold the section to the estimated equilibrium shape. It was anticipated that dredged material would be disposed in dike fields and on the convex bars (point bars) within the river channel. This placement would provide further contraction to the main-channel flow and encourage deepening of the channel by scour of the bed. The selection of disposal locations was based on the 1962 hydraulic model study of a 10 mile reach of the Arkansas River previously cited (Corps of Engineers, Waterways Experiment Station, 1962). It was concluded from this study that "dredged spoil and willow growth on convex bars improve the alignment, depth, and shape of channel through the bend. Such spoil banks require only minor protection at the upstream end" (Madden, 1964).

With the Arkansas project, dredging would be used primarily in a development role to accelerate natural morphologic processes; however, some maintenance dredging was also anticipated in the early years of the project because of local shifting of material into the dredged channel. Maintenance dredging requirements were expected to diminish with time as degradation continued, as a stabilizing armor layer of coarse material developed, and as the supply of sediment in the stream diminished.

On the Lower Mississippi dredging has been used extensively in a development role to support the river stabilization program. Carey (1966) termed the improvement program on the Lower Mississippi "comprehensive river stabilization" and considered channel dredging to be one of three primary construction techniques available to implement such a program. The two additional techniques were: preparing the river banks and bed for bank protection, and placing the bank protection. A fourth stabilization technique, contraction works, was considered secondary and Carey suggested the possibility "that massive corrective dredging can do directly, promptly, and with certainty what contraction works do indirectly, belatedly and with uncertainty." The concept of comprehensive river stabilization involves improving a river's alignment by an extensive cutoff (of meander loops) and corrective dredging program to produce a gently sinuous river, then fixing that alignment by bank protection. Carey applied this concept in retrospect to the stabilization program which began on the Lower Mississippi in 1928, but points out that the greatest possibilities for comprehensive river stabilization exist today on the unimproved alluvial rivers in the underdeveloped areas of the world. NEDECO's work on the Niger, Benue, and Rio Magdalena certainly supports this contention.

The major item of equipment for all phases of comprehensive stabilization, up to the placing of bank protection, is the hydraulic cutterhead dredge. In regard to dredging operations Carey observed that it should be the purpose of all dredging to give a "double effect," that is, the disposal of dredged material should be incorporated into the overall project plan so as to supplement the dredged cut. Where some riverine dredging is a "planless shoving of material from one place to another," the intent of double effect dredging is to move material out of the low-water prism where it creates a problem and dispose it so as to direct or confine the flow at higher stages.

Among the dredging operations that support a program of comprehensive river stabilization, the dredging of a pilot channel for cutoff of a meander loop clearly casts dredging in the role of a morphologic agent. While cutoffs occur naturally in a meandering stream, dredging can be used to greatly accelerate the process. A dredged cutoff is intended to reduce the length and curvature of a bend and produce lower stages for all discharges. A cutoff is usually made near the root of the meander loop (Figure 20) on a curve in the same direction as the original meander bend but with greatly reduced central angle and length, and increased radius. The geometry of the upstream and downstream bends is similarly altered by the cutoff. One approach is to use a cutterhead dredge to make a revetment cut along the line of the intended final concave bank and to revet it under slackwater conditions. The pilot cut for the actual cutoff is then excavated by the dredge on a curve parallel to the revetment cut, but toward the future convex bank. When this pilot cut is opened at the upstream end, natural processes of erosion enlarge the cut toward the revetment.

Dredging in support of the cutoff program on the Lower Mississippi between 1929 and 1945 produced significant morphologic change. The distance between the mouth of the White River and the mouth of the Red River was decreased by 96.7 river miles in this time period

(Carey, 1966). For a similar flood discharge of 1,500,000 cubic feet per second in 1929 and 1945 the average reduction in stage at eight gaging stations between the White River and the Red River was 11.1 feet on the rising portion of the flood hydrograph and 5.91 feet on the recession.

Reference to the dredging of cutoffs on the Lower Mississippi is intended to highlight the use of dredging as an agent for geomorphic change. While it is not the purpose here to argue the benefits of the cutoff program, it should be pointed out that there are "two schools of thought" on this subject (Winkley, 1971). One contends that cutoffs reduce flood heights, improve navigation channels, and do not cause adverse changes in river regime. The other holds that cutoffs intensify channel stabilization problems by increasing water surface slopes and velocity, causing excessive bank failure, and, in general, upset the equilibrium of the river. Besides changing the slope of a river, cutoffs also disrupt sinuosity and in turn, the sequence and spacing of pools and crossings. The controversy over the effects of dredged cutoffs and the significant changes that have resulted from the comprehensive river stabilization program underscore the potential of the dredging process as means of inducing change in a river system.

5.4.2 Gravel Mining

In regard to the geomorphic impact of dredging on the riverine environment, one application of the dredging process, dredging to obtain gravel for construction and related uses, is neither development nor maintenance oriented. Although not generally applied on the same scale as maintenance or development dredging, gravel mining can exercise a significant influence on a river system. Alluvial rivers have been a source of sand and gravel for many decades, but the impact on river morphology has only recently been recognized. On some European rivers, for example, the mining of gravel is now closely controlled by the agencies responsible for flood control, navigation, and river stability. Coarser materials are removed from the river only after due consideration has been given to possible adverse effects. Where gravel mining is permitted, improvement of the reach of river is the primary consideration, and the obtaining of gravel only a secondary benefit (Simons, 1973). The supply of gravel is currently being exhausted in many reaches of the Lower Mississippi River. Where the mining of gravel exceeds the natural rate of supply, detrimental changes in river morphology can result.

The impact of gravel mining is closely related to the role played by the coarser fraction of the bed material in controlling and stabilizing channel patterns and bed forms. This coarser fraction, particularly gravel, has a tendency, through hydraulic sorting, to armor the bed, thereby retarding or arresting excessive scour, stabilizing banks and bars, and preventing excessive sediment movement. Gravel armored sandbars can serve as semipermanent channel controls that define river form. Removal of the gravel armor from such features can lead to erosion and loss of this control. As a result, meandering reaches may tend toward a braided character, velocity and bed-material transport may increase, and localized changes may contribute to the deterioration of adjacent reaches (Winkley and Harris, 1973).

The armoring process has been described in detail by Livesey (1963). The process begins as the nonmoving coarser particles segregate from the finer material in transport. The coarser particles are gradually worked down into the bed where they accumulate in a sublayer at the level of the troughs of the bed forms moving through the system. This generally represents the lowest level to which the bed is turned over by the movement that accompanies the transport process. Fine bed material is leached up through this coarse sublayer to augment the material in transport. As movement continues and degradation progresses, an increasing number of nonmoving particles accumulate in the sublayer. This accumulation interferes with the leaching of fine material so that the rate of transport over the sublayer is not maintained at its former intensity. Eventually, enough coarse particles accumulate to shield, or "armor" the entire bed surface. When fines can no longer be leached from the underlying bed, degradation is arrested. Inhibiting this armoring process below a lock and dam was one of the objectives of dredging relative to the Arkansas River project.

Examination of typical armor layers reveals several important characteristics (Livesey, 1963):

1. Less than a single complete covering layer of larger gravel particles seems to suffice for a total armoring effect.

2. A natural "filter" apparently develops between the larger surface particles and the subsurface material to prevent leaching of the underlying fines.

3. The shingled arrangement of surface particles is not restricted to the larger material but seems evident throughout the gravel gradation.

It is evident that if a single layer of coarse material is sufficient to establish a bed armor, then armoring can develop quite readily. It is also apparent that armor layers will tend to accumulate in areas of natural scour in the river, such as on regions of the bed experiencing degradation and on the upstream end of islands and bars. Dredging for gravel is generally concentrated at these locations where the material is readily available.

Winkley and Harris (1973) present several examples that indicate the magnitude of the gravel mining problem and its effects on the Lower Mississippi. Assuming that a one-half inch particle is nonmoving under existing flow conditions and only one percent of the material underlying the bed surface is greater than one-half inch, then the depth of scour necessary to accumulate a single surface layer of one-half inch particles would be 0.5 inch ÷ 0.01 = 50 inches, or only about 4 feet of degradation. While this assumes that all particles greater than 0.5 inches remain in the region of scour, it is still a conservative estimate because of the influence of particle shape and the observation that the bed need not be completely covered with armoring gravel to produce an armoring effect.

Records in the Vicksburg District show that an average of 560,000 tons of coarse material is removed from the 166 miles of river in the District each year. Assuming that the upstream nose of a typical bar or island is 2,000 feet wide and 5,000 feet long and is armored with a one-inch layer of coarse material, less than 45,400 tons (33,600 cubic yards) of gravel could stabilize the nose of the bar. On this basis enough gravel is removed from this section of the river by dredging each year to stabilize 12 major bars or islands.

The impact of dredging gravel from the river is aggravated by other aspects of man's activity on the Mississippi. There are generally three types of gravel deposits along the Mississippi River (Winkley and

Harris, 1973):

1. Gravel deposits from the original alluvial valley fill sufficiently near the surface that the river frequently scours into them.

2. Reworked gravel deposits in certain meander belt areas.

3. Gravel deposits brought in by tributary streams.

In the past, the Mississippi frequently scoured or migrated into these gravel deposits. The gravel was usually transported downstream for a short distance to the head of a point bar or island where it remained until scoured by an unusually high flow or by migration of a meander bend into the bar formation. In today's river the bends and banks are generally stabilized by revetment or dike fields. Lateral migration of the river is restricted, and the river is seldom able to cut into any new gravel sources. Under these conditions the only sources of gravel available to the river are the bed, the upper end of bars or islands, and the generally small quantities transported in by tributaries. Gravel dredged from these locations is not readily replaced and the coarser materials are being depleted from the system.

The average particle size of bed material in the Vicksburg District changed significantly between 1968 and 1971. The median grain diameter, d_{50}, decreased by 41 percent and the d_{84} size (the size for which 84 percent of the material in the bed is finer) decreased by 34 percent. At one location in the Vicksburg District where gravel permits had been issued for several years the results of 128 bed samples between 1968 and 1972 showed a decrease of percent gravel in the bed from 26 percent to 4 percent (Winkley and Harris, 1973). When the importance of the coarser faction of the bed material is considered, changes of this magnitude must be expected to impact river morphology.

Formation of a gravel armor layer will tend to retard degradation of a riverbed and thus limit the depth of scour. Armor on the upstream nose of a point bar will resist formation of a chute channel and the development of a divided reach. Through examination of gravel dredging permits issued by the Vicksburg District, changes in flow through chute channels of divided reaches can be directly related to the time period during which gravel mining was allowed at the upstream end of a bar. Of the five divided reaches studied, Victoria Bend is representative (Figure 90). Divided flow conditions at Victoria Bend steadily improved between 1968 and 1971, with the chute channel receiving a smaller percentage of the flow each year. However, after 1971, when a permit was issued to dredge gravel from the reach, the divided flow situation deteriorated and the percentage of flow in the chute increased. A similar correlation of gravel mining and deterioration of the divided reach was established in four similar reaches of the Vicksburg District. Records in the District indicate that, at least partially due to the decreased bed material size, the channel has more divided flows, deeper revetment toe scour, and wider, shallower cross sections.

Because of the distribution of particle size with distance along a river system (Figure 13), the impact of gravel mining can be expected to be more severe in the lower reaches of the river. In the Upper Mississippi River basin (above Cairo) sand and gravel are available in great quantities from the glacial drift that covers most of the basin. These deposits are mined by both surface excavation and dredging. In 1950 more than 47,000,000 short tons of sand and gravel were removed from surface and riverine deposits combined. By 1960 production had almost doubled to 92,000,000 short tons, and it is expected that the production of sand and gravel will more than quadruple by the year 2020 (Upper Mississippi River Basin Coordinating Committee, 1972). As on the Lower Mississippi, river stabilization and revetment limit the upper river's source of gravel to deposits between the stabilized banks and materials introduced by tributaries. It must be anticipated that continued dredging of gravel from the Upper Mississippi on the scale projected will influence river morphology. Along the entire Mississippi system, then, dredging of gravel does constitute an agent for morphologic change which can and must be controlled by man.

5.4.3 The Lateral Redistribution of Sediment

Dredging has been viewed as an agent for morphologic change both in support of engineering requirements for river system development and as a consequence of man's continuing need for engineering construction materials. In each case dredging has significantly altered river characteristics. Each of these applications is overshadowed by the volume of material moved and the number of reaches involved in dredging operations for navigation channel maintenance. Dredging and open water disposal of dredged material in support of channel maintenance implies the moving of alluvial sediments from the main channel or thalweg region toward the periphery of the channel. This continued lateral redistribution of sediments, with some reaches on the Upper Mississippi being dredged 10-15 times in a 25 year period, not only interrupts the natural downstream movement of sediment in a system but also affects local channel morphology. In this section the impact of dredging and disposal on local channel morphology, in particular channel shape, is considered. Then the influence of channel shape on the transport of sediments through the system is examined.

The analysis of dredged cut stability indicates that even under optimum conditions of configuration and alignment a dredged cut in alluvium is only marginally stable. Investigation of test dredging, model studies, and the influence of stage on the crossing and pool sequence confirms the classification of dredging as a temporary means of improving navigation conditions, usually requiring repeated applications to maintain long-term channel integrity. In this regard the experience of the Corps of Engineers Mobile District in maintaining navigable depths on the Apalachicola River is typical (Odom, 1966).

The Apalachicola is formed by the confluence of the Chattahoochee and Flint Rivers at Chattahoochee, Alabama and from there flows 112 river miles to the Gulf of Mexico. The existing navigation project, which was approved in May 1953, provides for a 9-foot deep, 100-foot wide channel for the Apalachicola and portions of the Chattahoochee and Flint Rivers, with the channel on the Apalachicola to be maintained by supplemental channel work (dredging and contraction).

Establishing the initial 9-foot channel in 1957 required dredging 2,500,000 cubic yards of material. It was estimated at the time that 450,000 cubic yards of maintenance dredging would be required annually; however, maintenance dredging in 1958 exceeded 820,000 cubic yards and averaged 745,000 cubic yards between 1958 and 1963. In spite of the maintenance dredging in 1958, some reaches shoaled again before the year was out and portions of the river required dredging as many as three times during the low-flow season. By 1958 engineers of the

Mobile District concluded that it was "apparent that dredging alone would not provide navigable depths" on the Apalachicola (Odom, 1966). Because of these difficulties, a board of consultants was formed in August 1958 to advise the Mobile District on improvement and maintenance methods.

By 1963 a three phase schedule for improvement of the river had been developed, using contraction dikes, dredging, and improvement of a natural cutoff. Five dike fields were installed in 1963 under Phase I, and Phase II planning called for installation of 12 dike fields and one river cutoff. Permeable wood pile dikes with rock protection of the dike root and head were installed in 1963 and dredged material was placed in the dike fields. The significant point here is that experience from this project led to the conclusion that dredging "failed as the sole means for obtaining full project depth," and subsequent recommendations included the combined use of dredging and contraction dikes to maintain project dimensions."

The results of both the Manchester Islands hydraulic model and the model study of a typical reach of the Arkansas River cited previously support these recommendations. In view of this evidence as well as material to follow, it is difficult to concur in Carey's relegation of contraction works to a secondary role in a river development program (Section 5.4.1).

An excellent example of the effectiveness of the combined use of dredging and contraction works is provided by the Portland District's efforts to maintain navigable depths through the Henrici Bar on the Columbia River between Portland and St. Helens, Oregon (Kidby, 1966). One recommendation for maintaining a navigable channel on the

Columbia was to dredge and dispose of dredged material over-bank so that it could not find its way back into the channel, as shown schematically in Figure 91b. It was concluded that the increased cross section could reduce velocity sufficiently to decrease tractive force on the channel bed, and would, as a result, increase the rate of shoaling in the channel. An alternate concept involved the use of contraction dikes as in Figure 91c to help maintain velocities within the channel area itself and to reduce velocities near the banks. Dredged material from the main channel would be deposited in the dike fields along the periphery of the channel so that the total cross-sectional area of the channel remained as it was in the natural river, approximately the size required to move incoming sediments through the reach. A significant change in channel shape, indicated by a decrease in the width to depth ratio (W/D), is apparent between Figures 91a and 91c. This lateral redistribution of dredged material from the low-water prism to a location where it serves to confine or direct the flow at higher stages is a direct application of the concept of double effect dredging (Section 5.4.1). Disposal of dredged material in the dike fields rapidly accelerates the natural processes of deposition in the low-velocity regions between the dikes. The full impact of contraction works is felt by the channel much earlier than under natural conditions, and the dikes in turn provide a stability to the dredged material not possible at an unprotected disposal site.

The results of this process on the Henrici Bar are striking. Figure 92a shows soundings taken on the bar in 1909, before annual maintenance dredging was begun, and before contraction dikes were constructed. Under these conditions, dredging of more than 9000 feet of channel across the bar would be required to establish the 1960 project depth of 35 feet. In places the channel was as shallow as 8 feet before dredging. Attempts to maintain a 30-foot deep by 300-foot wide channel through this bar by dredging alone resulted in average annual dredging of nearly 700,000 cubic yards. During several years, more than one million yards of maintenance dredging was required.

Figure 92b shows the Henrici Bar reach in 1959. The first dikes in this reach were constructed in 1918 and were permeable pile dikes protected by a rock blanket. The hydrographic survey of Figure 92b taken before the 1960 dredging season, shows over 35 feet of water available in all but a few small areas of the channel. As a result, the dredging necessary to maintain a 35-by 500-foot channel on the bar now averages about one sixth of what was required to maintain a smaller channel without contraction dikes, an annual reduction of roughly 500,000 cubic yards of dredging. Dredging on the Henrici Bar during fiscal year 1963 was only 18,720 cubic yards.

The change in river cross section at the Henrici Bar between 1909 and 1959 is shown in Figure 93. The 1909 natural width of 4000 feet was decreased to 2300 feet 1959. Using width to depth ratio as an indicator of channel shape, the ratio of width to depth for this section decreased by more than a factor of three during this period. In summary, the Portland District's engineering concept for the 35-foot by 500-foot navigation channel includes stabilization of the channel and reduction of maintenance dredging to a minimum by the construction of pile dikes where needed, and placing dredged material between the pile dikes. This program has resulted in a significant change in channel shape and greatly reduced long term dredging requirements.

A similar concept was followed during the early period of development on the Upper Mississippi River. Contraction dikes were an integral part of both the 4 1/2-foot and 6-foot channel navigation projects during the period 1878 to 1930. Following the development of hydraulic pipeline dredges in 1895, dredging was used for navigation channel maintenance. Because of the concurrent application of contraction dikes, dredging, and dike field disposal, the geomorphic analysis of Pools 24, 25 and 26 on the Upper Mississippi provides an opportunity to investigate in detail the impact of the lateral redistribution of sediments on channel shape.

The geomorphic analysis of Chapter 4 revealed a general tendency toward aggradation and a decrease in river width in all three pools of the detailed study area between 1891 and 1930. Between 1930 and 1940, however, a lowering of bed elevations occurred in all three pools. This reversal has been attributed, in part, to the limited effectiveness of dikes constructed during the 4 1/2-foot project and early phases of the 6-foot project, as well as to the concentration of effort in the latter part of the dike construction era. Examination of the dredging records suggests an additional contributing factor. It was not until 1930 that annual dredging quantities in any of the three pools exceeded 300,000 cubic yards. Between 1930 and 1939, however, dredging quantities averaged 322,000 cubic yards per year in Pool 24, 650,000 cubic yards per year in Pool 25, and 1,320,000 yards per year in Pool 26.

The combined effect of dikes and dredging can be evaluated by comparing change in river cross section between 1891 and 1940. Geomorphic data for seven cross sections at locations where construction of dikes has been supplemented by dredging and disposal in the dike fields has been summarized in Table 17. In each case the cross-sectional area and top width for 1891 and 1940 were measured at the 1891 stage. Average depth (area/top width) and width to depth ratio are also shown. The change in width to depth ratio for these sections is striking. On the average, width to depth ratios decreased by 53 percent between 1891 and 1940, at these seven sections.

The Crider Island section at RM 280 in Pool 24 is typical of these cross sections. During the early 1880's (Figure 94) Dikes 3 and 4 had been placed across the chute channel to the east of Crider Island to divert flow toward the main channel. In 1919, Dike 7 was constructed in the main channel opposite Crider Island, and revetment was placed along the west side of the island to prevent erosion as the channel narrowed and the thalweg shifted in response to the dike. In 1924 and 1928, Dikes 8 and 11 were constructed to complete the diversion of the thalweg and contraction of the section. In addition to these contraction works the following quantities of material were removed from this reach by dredging: 1919--42,000 cu yd, 1923--15,000 cu yd, and 1935--22,000 cu yd. The response of this reach to these activities can be seen by comparing the change in cross section between 1891 and 1940 as shown in Figure 95. The closure of the chute channel by dikes 3 and 4 and the contraction resulting from dikes 7, 8 and 11 can be seen. The scour along the west bank of Crider Island which necessitated revetment construction is also evident. A 13 percent decrease in top width was accompanied by a 78 percent increase in average depth. As a result, width to depth ratio decreased by 51 percent, approximately the average percent decrease for the seven sections analyzed.

In contrast, geomorphic data for two reaches at which dredging and contraction efforts were not conducted jointly is shown in Table 18. The Middleton Island reach has no main channel dikes, but dredging between 1923 and 1936 removed 57,000 cubic yards from the main channel. The width to depth ratio in this reach increased by 64 percent between 1891 and 1940. In the Angle Island reach main channel dikes were constructed along both river banks, but dredging was not required between 1918 and 1940. Here, channel width to depth ratio decreased between 1891 and 1940, but only by four percent as compared to the average decrease of 53 percent at sections where dredging accompanied contraction efforts.

The engineering activities of contraction and dredging combined have a marked influence on channel morphology, in particular channel shape. The repeated displacement of bed material from the main channel region to the channel periphery is an obvious consequence of dredging. Also, any decrease in water surface slope over a crossing as a result of dredging would tend to reduce the movement of sediments from the crossing to the downstream pool. Each of these effects interrupts or retards the general downstream movement of sediment in a river system. The impact of changing channel shape on the transport of sediment through an alluvial reach is now considered.

In analyzing any hydraulics problem the uniform flow of clear water in a rigid boundary channel generally offers the simplest point of departure. For these conditions it is known that the conveyance of a channel section increases with an increase in hydraulic radius or with a decrease in wetted perimeter (Chow, 1959). As discussed relative to dredged cut stability, the channel section having the least wetted perimeter for a given area has the maximum conveyance. Such a section is known as the best hydraulic section. A semicircle has the least perimeter among all sections with the same area, and is, then, the most hydraulically efficient of all sections. For a rectangular, rigid boundary channel a width equal to twice the depth produces the best hydraulic section.

If the rigid boundary assumption is dropped, the problem becomes one of developing criteria for a stable, clear water channel in erodible material. Here, the change in velocity distribution with changing channel shape provides an indicator of the best channel section. Lane's (1937) summary of the work of Darcy and Bazin on this subject indicates the influence of channel shape on velocity distribution. Figure 96 shows velocity distributions in a number of rectangular channels having the same cross-sectional area but different width to depth ratios. Velocities are plotted as isovels, or lines of equal velocity, expressed as a percentage of the mean velocity.

An examination of the velocity distribution in these sections indicates that high velocities extend closer to the sides in the deep, narrow cross sections (low width to depth ratio) and extend closer to the bed in the wide, shallow sections (high width to depth ratio). The design of a stable clear water channel in erodible material requires the selection of a cross section with velocities that will not scour material from either the bed or banks. Since the material on the banks is acted on by gravity as well as fluid forces, the velocity required to move bank material is usually lower than that required to move bed material. Thus, to obtain the maximum possible mean velocity without scour, the velocity along the banks must be sufficiently less than the velocity along the bed to offset the gravity effect. This reduction in bank velocity relative to bottom velocity can be obtained by increasing the width to depth ratio (Lane, 1937).

If the assumption of clear water transport is replaced by the requirement to carry bed load, further modification of channel shape must be made. The transport of contact load is related to bed shear and velocity (Equations 25 and 35). In comparison to a stable, clear water channel, a channel carrying bed load must retain approximately the same velocity along the banks but have a higher velocity along the bed. This can be obtained with a wider, shallower channel. Therefore, heavily loaded channels in easily scoured material should have high ratios of width to depth (Lane, 1937).

This conclusion is in consonance with Schumm's (1963) classification of alluvial channels based on the dominant mode of sediment transport (Table 4). A stable suspend-load channel (less than three percent bed load) generally has a width to depth ratio of less than 10. A stable mixed-load channel (3-11 percent bed load) has a W/D ratio between 10 and 40, and a stable bed-load channel (greater than 11 percent bed load) exhibits a W/D of greater than 40. Investigations by Mackin (1948) and Wilcock (1971) also support the contention that "the broad, shallow channel is the type of cross section best adapted for the transportation of heavy bed load."

As has been noted, bed-material load is transported both in suspension and in contact with the bed (Section 3.2.4.2). These two modes of transport follow different physical laws, the former a turbulence phenomena and the latter a tractive force or shear related phenomena. In general the amount of bed material moving in contact with the bed of a large sand bed river is only a small percentage of the bed material moving in suspension. Leopold and Maddock in their classic (1953) paper recognized this difference, and in fact found that the relation between suspended load and channel shape is just the reverse of that between contact load and channel shape. At constant velocity and discharge, an increase in width (and thus, a decrease in depth) "is associated with a decrease in suspended load and an increase in bed load in transport" (Leopold and Maddock, 1953).

Gilbert (1914) investigated the relationship between transport capacity and form ratio (the ratio of depth to width) and concluded that there is an optimum form ratio for which a stream has its greatest capacity for transport. In a small, rectangular laboratory flume these values ranged from .5 to .05 (or in terms of width to depth ratio, from 2 to 20) depending on shape, discharge, and sediment size. Rubey (1952) rearranged Gilbert's transport capacity equation to the form:

$$S^a \left(\frac{D}{W}\right) = \frac{k\, Q_s^b\, d_{50}^c}{Q^e} \tag{44}$$

where:

S = graded slope of the stream or water surface

(D/W) = optimum form ratio

Q_s = bed-material load

d_{50} = average diameter of particles that make up the load

Q = volume of water discharged

k, a, b, c, e = constants.

When written as a proportionality, and after rearranging the variables, this relation closely resembles Lane's relation (Equation 11) with the added parameter of width to depth ratio:

$$Q\,S \propto Q_s\, d_{50}\, \left(\frac{W}{D}\right) \tag{45}$$

This relationship provides a convenient basis for a qualitative analysis which includes the effect of channel shape. Here, Q_s which represents bed-material transport, most of which moves in suspension, should be considered representative of suspended-load transport. On this basis, and assuming constant water discharge, slope, and particle size, an increase in width to depth ratio is associated with a decrease in suspended-load transport and vice versa, which is precisely the conclusion reached by Leopold and Maddock.

The lateral redistribution of sediments that results from the process of dredging in the main channel and disposal along the periphery of the channel can markedly alter channel morphology in a reach. Although dredging alone seldom accomplishes a long term change, the combined processes of channel contraction, dredging, and disposal of dredged material in the dike fields can significantly reduce the width to depth ratio in a channel section. Dredging deepens the channel, and disposal in a dike field greatly accelerates the natural processes of accretion there, while the dikes themselves lend a stability not possessed by unprotected disposal sites. The lateral displacement of material interrupts the natural movement of sediments, particularly contact load, through the system. The best hydraulic section for the transport of contact load is a wide shallow section (large width to depth ratio). Conversely, a narrower, deeper section (small width to depth ratio) is more efficient for the transport of suspended load. To the extent that dredging and contraction reduce the width to depth ratio at a section, they increase the channel's capacity for transport of the suspended portion of the bed-material load but decrease the capacity for transport of contact load. Thus, the lateral redistribution of sediment by dredging, when combined with contraction works, constitutes an agent for morphologic change in a river system. Physical displacement as well as the change in channel shape both retard the movement of bed-load sediments through the system, and it is as bed load that the bed material exercises its greatest influence on river form, character, and resistance.

5.5 Dredging and the 9-Foot Channel Project

The analysis of geomorphic data in Chapter 4, when compared with dike construction and dredging locations in the detailed study area, has indicated that contraction efforts and dredging at the same location can significantly alter channel shape, and thus sediment transport capacity. In this section geomorphic data is combined with an analysis of the dredging records for the detailed study area of the Upper Mississippi and for the Middle Mississippi to determine those factors that have exercised the strongest influence on dredging requirements in support of the 9-foot channel project. In addition, contrasts are drawn between dredging requirements in a series of regulated pools as on the Upper Mississippi, and under conditions of open river regulation as on the Middle Mississippi.

5.5.1 Analysis of the Dredging Records

For the Pool 24, 25 and 26 study area on the Upper Mississippi, dredging data has been compiled by the Rock Island and St. Louis Districts since 1906. Dredging volumes by year for each pool are summarized in Figures 97, 98 and 99, respectively. Annual dredging quantities range from a minimum of no dredging in some years to a maximum of 1,056,000 cubic yards in Pool 24 in 1938; 2,953,000 cubic yards in Pool 25 in 1933; and 4,119,000 cubic yards in Pool 26 in 1933. Dredging was not performed in any of these three pools between 1939 and 1948. Average annual dredged volumes before and after lock and dam construction are shown in Table 19. While dredging quantities remained essentially constant in Pool 26, average annual dredged volume increased by 66 percent in Pool 25 and decreased by 22 percent in Pool 24 after construction of the locks and dams.

Accumulated dredging quantities by five-year periods since 1950 indicate the trend in dredging requirements for each pool and provide a basis for projecting dredging requirements for the 1975-1985 period (Figures 100, 101 and 102). In each pool the minimum volume of dredged material moved was in the 1955-1960 period. Dredged volumes fluctuated in Pool 24, but the maximum volume during the 25 year record was during the last five-year period (1970-1975). In Pool 25 peak dredging was accomplished between 1965 and 1970 and then dropped slightly during the last period of the record. Pool 26 dredged volumes increased steadily after 1960 and reached a maximum in the last five years of the record. Using the five-year accumulations as a base, dredging quantities for the 10 years, 1975-1985, can be projected. These estimates show that approximately 1,000,000 cubic yards of dredging will be required between 1975 and 1985 in Pool 24. The projected requirement for Pools 25 and 26 is approximately 4,000,000 cubic yards in each pool. The detailed study area could require as much as 9,000,000 cubic yards of dredging in the next 10 years.

To determine if a trend exists when dredging volume is compared with river discharge, Figure 103 shows cumulative annual dredged volumes in Pools 24, 25 and 26 plotted against cumulative mean annual water discharge at Keokuk, Iowa, 64 river miles above the detailed study area. The relationship is broken into three distinct segments (1949-1954, 1960-1967 and 1969-1973) by years with little or no dredging (1955, 1957, 1959 and 1968). Between 1949 and 1954 the slope of the best fit line is 1.0, indicating that the dredged volume per unit mean annual discharge remained constant during the period. The last two segments of this plot, however, reveal an increased volume of dredging for a given volume of water since 1960. Dredged volume per unit mean annual discharge has doubled in comparison to the 1949-1954 period.

Establishing a direct cause and effect relationship between dredged volumes in the three pools of the detailed study area and geomorphic or hydraulic parameters is difficult. Not only do the number of engineering variables influencing dredging requirements cloud the issue, but such factors as funding and dredge plant availability also influence the quantity of material dredged in a given year. For example, dredging quantities in the St. Paul District by the Dredge <u>Thompson</u> began a sharp decrease in 1955. This decrease was not related to a geomorphic trend, but rather to assumption of Rock Island District dredging requirements by the <u>Thompson</u>. These additional dredging requirements resulted in postponing some St. Paul District dredging and produced the decrease in the record of dredged volumes between 1955 and 1964 (Corps of Engineers, St. Paul, 1974). Operational policies such as the practice of overdepth or overwidth dredging also influence dredged quantities, and consequently, raise questions concerning the extent to which dredged volumes represent actual dredging requirements in response to conditions on the river. Considering these limiting factors, it is still possible to advance several reasonable hypotheses concerning trends apparent in the dredged volume records of the detailed study area.

The large dredging volumes of the 1933-1938 period (Figures 97, 98, 99) can be related primarily to dredging associated with the transition from the 6-foot channel project to the 9-foot channel project. Development of the 9-foot channel required moving a large stockpile of sand not touched by efforts to establish the 6-foot channel. Moving this stockpile of bed-load material required several years to accomplish. In addition, the mid-30's were years of extremely low flows in the detailed study area. Between 1931 and 1937 low stage at the Hannibal gage (Figure 59) was below the gage zero in six out of seven years. Although dredging studies by the Memphis District (Section 5.3.4) showed that a prolonged low-water period can improve navigation depths, the required low-water scour on the crossings presupposes sufficient water in the channel to redistribute sediments deposited during higher flows. With the extreme low water of the 1930's, the lack of water in the channel apparently became the controlling factor, and dredging of larger volumes was required to maintain navigable depths.

The break in the dredging record between 1939 and 1948 can be attributed, in part, to the depletion of the large stockpile of bed-load sediments by the heavy dredging of the 1930's. However, the turbulence of the war years and resulting reordering of priorities probably had more impact than geomorphic or hydraulic factors on the lack of dredging during this period. Moreover, this is the approximate period in which the results of bank stabilization and soil conservation measures instituted in the mid-1930's would be felt. Conservation, stabilization, and impoundment of tributary streams all reduce sediment yields at the primary sources and thus reduce the quantity of bed-material sediments reaching the navigation channel.

The increasing dredging quantities during the 1960's can be partially attributed to a period of unusually high flows. During the 70 year period 1903-1973 the Hannibal gage (Figure 59) has exceeded a 22-foot stage on nine occasions. Four of these nine occurred between 1960 and 1973, with the highest stage of record (28.59 feet) in 1973. The effect of high flows on the crossing and pool sequence is one of scour from the pools and deposition on the crossings. For a dredged cut oriented with the low-flow thalweg, high flows generally produce severe cross currents and resulting fill of the cut. The studies by the Memphis District in the early 1930's relate stage and dredging requirements (Figure 88) and clearly show that increased dredging requirements accompany periods of high flow.

The policy of overdepth and overwidth dredging could also contribute to the increase in dredged volumes in the 1960's. The effects of overdepth dredging have been examined (Section 5.3.1) and are apparent in the results of the model study of Lock and Dam 8 on the Arkansas River (Section 5.3.3). Here, it was concluded that increasing the size of the initial dredged cut would reduce the amount of short-term maintenance dredging but would tend to increase the total amount of dredging required when both the initial cut and long-term maintenance dredging quantities were considered. In addition, the longer barge tows of recent years have required reduced bendway curvature, and thus, increased point bar dredging requirements.

Just as an operational policy such as overdredging can influence dredged volumes, the effectiveness of the dredging operation can also contribute to a change in dredging quantities such as the increase in dredged volumes in the 1960's. In attempting to relate trends in dredged volume to geomorphic or hydraulic factors, the extent to which quantities dredged actually reflect requirements imposed by the river must be considered. In a review of research requirements on channel stabilization problems, Tiffany (1963b) concludes: "It is believed that a very careful analysis should be made of the dredging procedures used in the rivers of concern, since it is considered possible that at least some of the dredging is misdirected and/or ineffective." Both overdredging and dredging beyond the requirements imposed by the river could contribute to the doubling of dredged volume per unit discharge since 1960 indicated by Figure 103.

One aspect of the records of dredged volume that does appear directly related to geomorphic factors is the quantity of dredging performed in Pool 24 as compared to dredging in either Pool 25 or 26. Although Pool 24 is the shortest of the three pools (27.6 river miles as compared to 31.9 and 38.4 river miles for Pools 25 and 26), dredging in Pool 24 averaged 63 percent less than the average of Pools 25 and 26 between 1906 and 1938, and 77 percent less between 1949 and 1973 (Table 19). This difference cannot be attributed solely to length of reach. Geomorphically the most significant difference between the Pool 24 reach and the Pool 25-26 reach is in river width. The average river width in the Pool 24 reach was 4100 feet in 1927 (Table 5) as compared to an average of 5013 feet in the Pool 25 reach. In 1973 the river width in Pool 24 averaged 4000 feet, a decrease of 100 feet, while the river width in Pool 25 averaged 5776 feet, an increase of more than 750 feet. A comparison of average depths derived from cross-sectional data (Tables 7, 17, and 18) reveals that the river is not appreciably deeper in the Pool 24 reach than in Pools 25 and 26. Consequently, flow velocity and sediment transport capacity in the narrower reach of Pool 24 can be expected to be greater than in the two wider pools, resulting in decreased dredging requirements. The impact of an additional geomorphic factor, channel pattern, in influencing dredging requirements in Pool 24 is examined subsequently.

Development of the navigation channel on the Middle Mississippi has been by open river regulation, that is, by contraction dikes, revetment, and dredging and disposal in the dike fields. Without the influence of navigation locks and dams, as on the Upper Mississippi, the geomorphic response of the river to man's development has been less complex. As a result, the relationship between dredging quantities and geomorphic change is more clearly evident.

On the Middle Mississippi between river mile 0 at the mouth of the Ohio River and river mile 195 at the mouth of the Missouri River, dredging quantities by year since 1963 are summarized in Figure 104. Annual dredging quantities have declined from a peak of 8,131,000 cubic yards in 1965 to a minimum of 2,056,000 cubic yards in 1973. This decline can be directly related to contraction efforts for the 9-foot channel project. As outlined in Chapter 4 (Section 4.4.2), contraction dikes on the Middle Mississippi prior to 1944 were designed to constrict the river to widths ranging from 2500 to 2000 feet. In the post-war era an 1800-foot contraction plan was adopted, but by 1960 it was evident that this plan would not insure a low-flow navigation channel without supplemental dredging. After investigation of the effects of contracting a study reach (the prototype reach, Figure 6) to a 1200-foot width, the current 1500-foot width contraction plan was adopted. The effects of these contraction efforts are apparent in Figures 45, 62 and 63, and the impact on river widths is summarized in Table 9. The result has been generally decreasing dredging requirements on the Middle Mississippi.

Using the prototype reach from RM 140 to RM 154 as an example, average width in the reach was reduced from 4800 feet in 1889 to 1800 feet in 1966 and the riverbed was lowered by an average of 8.0 feet. Between 1967 and 1969 the prototype reach was narrowed to 1200 feet and the riverbed dropped an additional 3.0 feet. The effect of this man-induced geomorphic change on dredging requirements can be seen in Table 20 and Figure 105. Dredging requirements in the prototype reach between 1963 and 1974 are shown in Table 20. Dredged quantities increased to a peak of 932,000 cubic yards in 1966. Construction of the prototype reach began in July 1967 and was completed in March 1969. Dredged volume in the reach was reduced by 54 percent in 1967 and by another 38 percent in 1969. Since 1969 there has been no dredging performed in the 14-mile reach. Figure 105 shows graphically the effect of contraction efforts in the prototype reach on dredged quantities in comparison to upstream and downstream reaches of the Middle Mississippi.

The extensive contraction efforts on the Middle Mississippi have been accompanied by a major reduction in both width to depth ratio and flow area. At St. Louis (Figure 63) width to depth ratio has decreased by 63 percent and the cross-sectional flow area in 1973 was only two-thirds of the 1837 flow area. In addition, main line levees along the Middle Mississippi have reduced floodplain storage for flows greater than bankfull. The effect of a decrease in both flow area and overbank storage can be seen in Table 11 and Figure 66. Table 11 shows a reversal of the stage-discharge order in that floods of recent years with relatively low discharges have produced stages that are among the highest of record. The 1973 flood, for example, ranked only 10th in discharge but produced the highest stage of record (43.3) at St. Louis. Figure 66 shows the trend of changing stage for a given discharge on the natural river and on the developed river at St. Louis. Using this figure, it is estimated that the 1844 record discharge of 1,300,000 cfs which produced a stage of 41.3 feet at St. Louis would now pass St. Louis at approximately a 52.0 foot stage.

Experience with the prototype reach on the Middle Mississippi has shown that sufficient contractive effort can eliminate maintenance dredging requirements. However, contraction works on the Middle Mississippi have been responsible, in part, for a significant increase in stage at the higher discharges. When permanent improvements such as contraction of a river to obtain navigation depths produce undesirable hydraulic or geomorphic response, dredging provides an alternate means of assuring required depths for navigation. Using the Middle Mississippi as an example, contraction to minimize dredging can be applied only so long as increased stages are still acceptable. Beyond this point continued maintenance dredging provides an alternative to additional contraction and the consequent increase in flood stage. Similarly, accepting dredging as a trade-off or compromise solution to avoid undesirable secondary effects of more permanent means of insuring navigation depths was precisely the recommendation resulting from the Manchester Islands model (Section 5.3.3).

The records of dredged volume on the Upper and Middle Mississippi can also be used to provide an estimate of the magnitude of the dredging and disposal effort in relation to sediment moving through the system. The measured average annual suspended load at Hannibal of 20,400,000 tons/year is an indicator of sediment moving through the Pools 24, 25, and 26 study area. If the contact load is estimated at 10 percent of the measured suspended load or 2,040,000 tons/year, then the total load is approximately 22,440,000 tons/year in the detailed study area. The average annual dredged volume between 1949 and 1973 in the three pools was 867,000 cubic yards (Table 19), or approximately 1,170,450 tons/year. Dredged volume in the three pools, then, represents only about 5 percent of the total load moving in this reach, but about 57 percent of the contact load. In view of the impact of the lateral redistribution of contact load on channel morphology and considering the importance of contact load to the determination of river form and chracter, interrupting the down-system movement of more than 50 percent of the contact load in a 100-mile reach of river is significant.

On the Middle Mississippi at St. Louis the measured average annual suspended load is estimated at 160,900,000 tons/year. Using a factor of 10 percent, bed load at St. Louis is approximately 16,900,000 tons/year and total load is 177,800,000 tons/year. Dredged volume for the entire Middle Mississippi averaged 4,252,460 cubic yards or

5,740,000 tons/year between 1963 and 1974. On the Middle Mississippi, then, dredging impacts about 3 percent of the total sediment load and as much as 34 percent of the contact load. This is, again, a significant portion of the bed load moving through the system.

5.5.2 Geomorphic Factors and Dredging Requirements

To establish a more direct relationship between dredging requirements and geomorphic factors it is useful to cast the dredging data in terms of dredging frequency by location. Here, dredging frequency is taken as the number of dredging events, regardless of volume, at a given location during a specified period. To simplify the analysis, locations are considered only to the nearest river mile. Dredging frequencies by location from river mile 200 to river mile 300 are shown in Figure 106 for the 1918-1938 pre-lock and dam period, and for the 1949-1973 post-lock and dam period of the dredging record. Also indicated on Figure 106 are the locations of the three locks and dams of the detailed study area, and tributary locations. In addition, the general morphologic character of the river at a given location in terms of pools, crossings, straight reaches, and divided reaches is also included.

Perhaps the most striking characteristic of Figure 106 is the difference in the distribution of dredging requirements before and after construction of the locks and dams. In the 21-year period, 1918-1938, under conditions of open river regulation, the maximum dredging frequency was nine dredging events (between RM 221 and RM 222), and all but 12 river mile locations required dredging. In the 25-year period of record following lock and dam construction peak dredging frequency was 15 dredging events (between RM 266 and RM 267), and seven locations experienced nine or more dredging events. In addition, there were 46 locations that did not require dredging during the period. The pools of the three locks and dams in the detailed study area radically altered the distribution of dredging requirements.

While the dredging frequency plot of Figure 106 tends to highlight dredging trouble spots, it also includes locations that have required only infrequent dredging. In addition, Figure 106 does not consider dredged volume which can be as strong an indicator of dredging problems as frequency. Accordingly, Figure 107 combines frequency and volume, and considers only those locations that can be classed as trouble spots using the following criteria:

Continuous Dredging (C)--	a location with a frequency of at least eight dredging events in the last 20 years (1954-1973) and at least one dredging event in the last five years
Recurrent Dredging (R)--	at least 5 dredging events in the last 20 years and at least 1 dredging event in the last 5 years
Recent (R_c)--	at least 2 dredging events in the last 5 years
Volume (V)--	a location which does not meet the above frequency criteria but which has required at least 500,000 cubic yards of dredging between 1949 and 1973.

With this combined frequency and volume criteria, dredging volumes at trouble spots during the 1949 to 1973 period are plotted in Figure 107. As with Figure 106, tributary, lock and dam, and pool control point (PCP) locations as well as the general morphologic character of the river are shown. The relationship between geomorphic factors and dredging requirements is investigated using Figures 106 and 107 as the primary references.

Contrary to what might be expected, dredging in the detailed study area does not appear to be strongly related to tributary locations. The two largest tributaries in the reach, the Salt and Illinois Rivers, are not associated with a dredging trouble spot or even infrequent dredging (Figure 106 and 107). The combined influence of four smaller tributaries, Bob's, Peruque, and Dardenne Creeks, and the Cuivre River, between RM 228 and RM 238 has contributed to a shift in the Mississippi from the western bankline toward the eastern bankline (Section 4.2.1). Moreover, Rubey (1952) attributed the formation of Peruque, Dardenne, and Cuivre Islands to these tributaries. Although the reach bracketed by these tributaries contains locations of recurrent, recent, and volume dredging, the relatively short time scale which must be considered with regard to dredging problems and the relatively small water discharge volumes of these tributaries both dictate against considering these tributary streams the primary contributors to dredging problems in the reach.

In other locations on the Upper Mississippi the evidence suggests that tributaries do exercise a major influence on dredging requirements. The Chippewa River, for example, is a source of a large amount of coarse sediment which contributes to dredging requirements as far downstream as Pool 5A (Figures 1 and 44). It has been estimated that the Chippewa is responsible for about 20 percent of all maintenance dredging along the Mississippi River between Pool 4 and Pool 10, 148 miles downstream (Corps of Engineers, St. Paul, 1974).

Both Figures 106 and 107 reveal a strong correlation between lock and dam locations and dredging requirements. A location requiring continuous dredging exists just downstream from Lock and Dam 22 at RM 298. The particularly troublesome RM 260 to RM 270 reach begins 4.4 river miles downstream from Lock and Dam 24, and a recent dredging problem exists just downstream from Lock and Dam 25. Figure 106 indicates additional small scale dredging requirements immediately above and below Lock and Dam 24, and just above Lock and Dam 25. Dredging requirements relative to lock and dam locations can be attributed to the classic pattern of accretion above a dam and clear water scour below (Figure 15). The longitudinal profiles through Lock and Dam 24 (Figure 51) for the 1930-1971 period show the tendency to accumulate sediments in the pool above a lock and dam and the development of a major scour hole below the lock and dam. This scour hole acts as a sediment source area and contributes to downstream sedimentation and dredging problems.

The influence of pool operations on conditions in the upper and lower halves of a navigation pool is described in Chapter 4 (Section 4.1.2). The three pools in the detailed study area are operated about primary control points located about halfway between navigation dams. The gates of the navigation dams are operated to insure navigable depths throughout the pool by maintaining a required minimum elevation at this control point. Only the area between the control point and the downstream dam is inundated by operation of the dam. The area between the control point and the upstream dam remains in an open river configuration except that low-flow stages are controlled to provide navigation depths. Primary control points for pools 24, 25, and 26 are located at Louisiana (RM 282.1), Mosier Landing (RM 260.3), and Grafton (RM 218.0), respectively, and are shown on Figure 107. Dredging trouble spots in Figure 107 tend to be located above the pool control point in each pool, where the river is flowing under essentially open river conditions. In Pool 24, 100 percent of the continuous, recurrent, recent, and volume dredging occurs between the primary control point at Louisiana and Pool 22, upstream. In Pool 25, 73 percent of the trouble spot dredging occurs above the primary control point, and in Pool 26, 100 percent occurs above the primary control point at Grafton. Only relatively minor amounts of dredging are required in the ponded portion of the pool below the primary control point. At present, water levels in this portion of the pool apparently provide depths in excess of the required navigable depth on most crossings and, thus, reduce dredging requirements.

Field experience has established that most dredging operations are conducted to improve depths on the shallow crossings or to increase channel width adjacent to point bars in bends. While the scale of Figure 107 does not permit a detailed correlation of dredging requirements with the crossing and pool sequence, it does permit a comparison of dredging problems in reaches where a definite crossing and pool sequence exists with requirements in essentially straight reaches. The unstable nature of straight reaches is described in Chapter 3 (Section 3.2.1.2). Where the thalweg meanders through a series of alternate bars in a straight reach, rapid and significant shifts in thalweg location can be expected as alternate bars grow, deteriorate, or move downstream. In relating the location of dredging trouble spots to straight reaches between RM 200 and RM 300, Figure 107 reveals that of the three locations requiring continuous dredging, all three are located in straight reaches. Of the eight locations of recurrent dredging, six are associated with straight reaches, and four of the seven recent dredging locations are in straight reaches. Finally, all four of the volume dredging trouble spots are located in straight reaches. On the basis of dredged volume, 85 percent of the dredging in the detailed study area is associated with straight reaches.

When evaluating the impact of straight reaches on dredging requirements an additional factor must be considered. Many straight reaches contain alluvial islands which produce the divided reaches shown on Figures 106 and 107. Reaches are classed as divided only if they contain large alluvial islands with relatively open chute channels, such as Gilbert Island (RM 298), Carroll-Slim Islands (RM 264-269), and Bolters-Iowa Islands (RM 222-226).

The morphology of divided reaches was outlined in detail in Chapter 3. Rubey's data (Figure 29) shows an increase in the relative depth (depth/width) when the individual channels of the divided reach are compared to the undivided upstream channel. This implies a decrease in the width to depth ratio which has been correlated with an increased capacity for suspended sediment transport but a decreased capacity for the transport of contact load. The measured patterns of water and sediment flow in the Long Island reach of the Niger River (Figure 30) point to significant sedimentation problems at the entrance to and exit from a divided reach (Section 3.2.3.3). This measured data on a natural river is supported by the results of the Manchester Islands model study of a typical divided reach of the Ohio River (Section 5.3.3). Maintenance dredging problems can be anticipated, then, when a navigation channel passes through a divided reach.

The dredging data from the detailed study area supports this conclusion. The trouble spot requiring continuous dredging just downstream from Lock and Dam 22 is located just above the Gilbert Island divided reach. The concentration of dredging requirements between RM 264 and RM 269 can be related to the division of a straight reach into multiple channels by the Carroll-Slim Islands complex. This reach of Pool 25 constitutes the most serious dredging problem area in the three pools and contains two continuous dredging locations, three locations requiring recurrent dredging, and one requiring volume dredging. In Pool 26 both recurrent and recent dredging trouble spots correlate closely with either the entrance to or exit from a divided reach.

To summarize, Figures 106 and 107 reveal a close correlation of dredging trouble spots in the detailed study area with the following factors:

1. Location of locks and dams.
2. Location relative to pool primary control point.
3. Straight reaches.
4. Divided reaches.

Combinations of these factors also influence dredging requirements. Between RM 200 and RM 300 the most serious dredging problems occur in straight reaches which are located above the pool primary control point, and which are divided by alluvial islands.

5.5.3 Contrasts

Under conditions of open river regulation as on the Middle Mississippi the relationship between geomorphic change and dredging requirements is quite clear. Using the 14-mile prototype reach as an example (Table 20), it is apparent that dredging requirements are evenly distributed through the reach, with only three river mile locations not experiencing at least one dredging event during the 11 year period of record. This pattern is quite similar to the 1918 to 1938 dredged frequency plot on the Upper Mississippi (Figure 106) under conditions of open river regulation.

Dredging volumes along the entire Middle Mississippi have decreased as the river was subjected to additional contraction. The result of carrying contraction efforts to the extreme can be seen in the prototype reach which has not required dredging since contraction to a 1200-foot width in 1968. Decreased dredged volumes under conditions of open river regulation on the Middle Mississippi, however, have not been attained without consequences. Hydraulic data at the St. Louis gage shows that contraction works have been at least partially responsible for significantly increased stages at the higher discharges. When open river regulation is used to insure navigable depths in a river system, there is evidently a limit to which contraction can be applied. Dredging provides a compromise solution to the problem of maintaining required navigation depths and at the same time avoiding the undesirable secondary effects of further contraction.

When the transition is made from open river regulation to low-flow regulation by a series of locks and dams as on the Upper Mississippi, a significant redistribution of dredging requirements can be anticipated

(Figure 106). Dredged volumes can increase markedly as in Pool 25 or decrease as in Pool 24. The redistribution of dredging requirements in the pools of a lock and dam system is not random, but rather is related to geomorphic characteristics of the pools. The locks and dams themselves become dredging problem areas. In particular, the reach downstream from a dam has a high potential for causing dredging problems. Local scour below a dam provides a source of sediment which is generally carried to the next crossing or divided reach downstream. Tributaries also provide a source of sediments and dredging problem areas can be anticipated where tributaries enter the ponded portion of a navigation pool. In this regard the evidence from the detailed study area is contradictory as tributary locations in the three pools do not coincide with dredging trouble spots. It should be noted, however, that both major tributaries (the Salt and Illinois Rivers) in the reach enter the navigation pools above the pool control point where the river is flowing under essentially natural conditions.

Water levels in Pools 24,25, and 26 are maintained relative to a primary control point located in the middle half of the pool. For navigation pools operated in this manner, evidence from the detailed study area suggests that dredging problems will be concentrated in the portion of the pool between the control point and the next dam upstream.

In regard to geomorphic factors, field experience has established that under conditions of either open river regulation or low flow regulation with navigation dams, the crossings in a meandering thalweg river are potential dredging problem areas. Analysis of the dredging data from Pools 24, 25, and 26 also establishes straight and divided reaches as potential dredging trouble spots. Figure 107 shows that the combination of a straight reach which is located above a pool control point and is also divided by large alluvial islands produces the most severe dredging problems. Since the river is flowing in an essentially open configuration above the control point it can be expected that straight and divided reaches will also represent regions of potential dredging problems under conditions of open river regulation. Where continuous or recurrent dredging requirements exist above the control point of a navigation pool, obtaining permanent channel improvement by contraction should be considered. Decreasing river width by contraction has successfully reduced dredging requirements on the Middle Mississippi and should also be effective in those portions of a navigation pool above the pool control point.

5.6 The Dredging Problem and Geomorphic Indicators: A Case Study

The geomorphic indicators of dredging problems which have been developed through an analysis of the response of a river to dredging provide the river engineer with a means of analyzing a variety of problems related to navigation channel maintenance. This section illustrates the problem solving potential of these geomorphic indicators, by describing their application to a current dredging problem on the Upper Mississippi River.

The Corps of Engineers, St. Paul District plans to initiate a pilot program of reduced overdepth dredging during the 1975 navigation season. During the 1974 navigation season, ten sites in the St. Paul District were dredged to depths less than the usual 13.0 feet below low control pool (Section 5.2.3). Although these sites were dredged to less than 13 feet for reasons other than hydraulic considerations, channel condition surveys were made at most of these sites before dredging, immediately after dredging, and several months after dredging. In February 1975 Colorado State University was furnished copies of channel condition surveys at these ten sites and a listing of those sites in the St. Paul District which were being projected for potential maintenance dredging during the 1975 navigation season. Comments were requested concerning the sites or types of sites which should be selected from the list of 1975 projected dredging locations for inclusion in the reduced overdepth dredging pilot program. Identification of those locations which would have the greatest probability of success in reducing the depth of maintenance dredging without incurring an undue risk of channel failure was desired.

The geomorphic indicators of dredging problem areas developed in this chapter were used to devise a procedure for selecting sites for possible inclusion in the reduced overdepth dredging pilot program. As a test, the procedure was applied to the ten sites dredged to reduced depths during the 1974 season. Channel condition surveys before and after dredging and the dredging frequency records were used to evaluate the selection process. If the geomorphic analysis resulted in recommending a site for inclusion in the pilot program which has required continuous or recurrent dredging or for which the post-dredging channel condition surveys indicate only marginal stability, then the selection criteria was considered invalid at that location. Conversely, if the geomorphic analysis recommended a site which has experienced only low dredging frequencies or which post-dredging surveys show to be reasonably stable, the selection criteria was considered valid. For the actual selection of dredging locations for inclusion in the 1975 pilot program, the records of dredging fequency by location provide an additional criteria for the selection process.

The geomorphic indicators used in the selection process and the results of this analysis are summarized in Table 21. The selection criteria included location of the site, type of reach, high-flow alignment, location of the cut (with respect to the thalweg), and channel conditions before dredging. Projected depth and projected volume of the cut as well as channel conditions after dredging are also shown. The location of the dredged site relative to the pool control point was not used since the dredging records in the upper pools of the navigation system do not exhibit the tendency for dredging problem areas to be located above the control point that is so apparent in Pools 24, 25, and 26. Location of a site just below or just above a lock and dam or in the vicinity of a heavy sediment carrying tributary such as the Chippewa River was considered a negative factor in the analysis. The straight, divided reach was considered the least desirable location for a pilot program dredged cut and an undivided bend the most desirable. High flow alignment was evaluated based on the assumption that the dredged cut would be located with regard to low-flow conditions. Locations where high flows could short circuit a bendway or bypass the main channel through a chute channel were considered undesirable since major changes in the direction of flow generally produce crosscurrents which fill the dredged cut. Location of the dredged cut in alignment with and on the thalweg was considered a positive factor, while location on or near a point bar where there is a readily available source of sediments to refill the cut was considered a negative factor. Locations with greater average depths before dredging were considered favorable since the before dredging depth is indicative of the natural depth that the flow can maintain in a reach.

Using these criteria, each of the ten locations dredged during the 1974 season was evaluated. The results are summarized in Table 21. Although there is a degree of subjective analysis in the selection process, the factors considered in Table 21 provide a reasonable basis for decision making. Of the ten sites analyzed, four were selected for the pilot program on the basis of geomorphic indicators. When compared with the records of dredging frequency, all four selected sites exhibited low dredging frequencies, ranging from three to six dredging events in the 19 year period of record. Surprisingly, post-dredging surveys show that all of the sites remained relatively stable. However, three of the locations not selected (Crats Island, Fisher Island, and Brownsville) experienced average depth increases of less than 1.0 foot immediately after dredging. Only one site, Winters Landing (Cut 3), tended to fill between the post-dredging channel condition surveys. Between July 1974 when most of the dredging was accomplished and November 1974 when the final condition survey was made, the Upper Mississippi had abnormally low flows. In view of the effect of high stage or fluctuating stage on dredged cut stability, this low-flow period could account for the apparent stability of the dredged cuts.

Of the sites not selected for the pilot program, only one, Beef Slough, had a low dredging frequency. The divided character of the Beef Slough reach, an average high flow alignment, a poor dredged cut location, and a shallow before dredging depth all dictate against selecting the location for a dredging pilot program. The low dredging frequency and increase in depth in the cut contradict the indicators in this case. The geomorphic analysis resulted in the selection of four of the five locations with low dredging frequency, and did not select any of the high dredging frequency locations for inclusion in the pilot program. The procedure is evidently quite conservative, and when applied to the list of 1975 projected dredging sites will constitute a reliable method for selecting sites for inclusion in the reduced overdepth dredging program.

5.7 Disposal of Dredged Material

The impact of dredged material disposal on river morphology has been investigated in relation to the effects of lateral redistribution of sediments from the main channel to dike fields on the channel periphery. In this section the impact of open water disposal of dredged material on the riverine environment is examined in more detail. Here, open water disposal includes dredged material placed on islands, marshes, and along riverbanks at locations where these materials are subject to the influence of river stage fluctuations, or are readily washed back into the river by rainfall. The impacts of primary concern are the processes of filling sloughs, chute channels and backwater areas, and the concomitant loss of diversified habitat in these biologically productive regions. A summary of the environmental impact of dredged material disposal is followed by an examination of the current practice of bankline and island disposal in relation to river morphology and hydraulics. Finally, the feasibility of disposing dredged material in the main channel region of the river to minimize environmental impacts is investigated using a mathematical model of the Pool 24, 25, and 26 study area.

5.7.1 Environmental Impacts of Dredged Material Disposal

Environmental impact statements prepared by the St. Paul District (1974) and the Rock Island District (1974) for the operation and maintenance of the 9-foot channel project provide an excellent summary of the effects of dredged material disposal on the riverine environment of the Upper Mississippi River. In addition, the Waterways Experiment Station has conducted a physical, biological, and chemical inventory and analysis of selected dredging and disposal sites on the Middle Mississippi River (Solomon et al., 1974). The following summary of the environmental impacts of dredged material disposal is extracted primarily from these three sources.

To understand the impact of dredged material disposal on the natural environment requires developing an understanding of the natural processes of the riverine environment. In the Mississippi River valley these processes have been disrupted in many areas by man's activities. Intense cultivation of the river floodplain has removed forest cover along much of the river, permitting large amounts of silt to enter the river and accelerating the accumulation of sediments in quiet backwater areas. The creation of navigation pools on the Upper Mississippi in the 1930's resulted in the establishment of large areas of aquatic habitat (Section 4.1.2). Eventually, portions of the aquatic habitat evolved into marshlands suitable for many species of birds and animals dependent on marshland vegetation for food and shelter. This successional change from aquatic habitat to marshland, however, resulted in a reduction of aquatic habitat suitable for benthic organisms of value to other forms of fish and wildlife.

The open water aquatic community consists of a benthic community, composed of plants and animals that live on and in the riverbed, and a pelagic community, composed of plants and animals that live within the water column. The kinds of organisms which comprise the benthic community depend upon the bottom substrate, which in turn is dependent on such physical parameters as sediment type and current velocity. Observations indicate an absence of benthic organisms in portions of the main river channel. Maintenance dredging may be responsible, in part, for this absence, however, the main channel bottom of the Upper and Middle Mississippi generally consists of a steadily moving sequence of bed forms (Figure 38) composed of rather sterile, sandy contact-load sediments which do not form a suitable substrate for the development of benthic organisms. The pelagic community of fish and floating organisms such as phytoplankton and zooplankton are strongly influenced by light penetration, water depths, and currents. Dredged material deposited in peripheral areas of the channel can affect both benthic and pelagic habitats by altering current velocity, reducing water depth, increasing turbidity, or changing the nature of the substrate.

Construction of dikes for the 4½ and 6-foot channel projects reduced the aquatic habitat by directing water into a single navigation channel; however, the rock fill used in the construction of these structures created prime aquatic habitat for many species of fish. In many cases, these habitat conditions continued to exist after creation of the navigation pools of the 9-foot channel project. In addition, the extent of aquatic habitat was greatly increased by the formation of the large navigation pools (see Figure 49 for example). Subsequently, this habitat has continually been reduced by the settling of fine sediment particles. The current practice of disposing dredged material in the dike fields and peripheral channel areas frequently accelerates this process of filling.

River marshes form a vital ecological transition zone between the open water aquatic habitat and terrestrial habitats. Marshes draw from and contribute to the ecosystems of both habitats. Since all life of the river system relates directly or indirectly to life in the marshes, the primary impact of dredged material disposal in a marsh is the direct elimination of extremely valuable habitat. Organisms in the marsh which cannot escape at the time of disposal will be smothered. Those able to escape will be forced to find new sources of food and shelter. During certain seasons, such as the spring or fall, when plant productivity is low and migratory waterfowl number in the thousands, the availability of suitable marshland habitat can become critical.

A secondary impact on marshland, sloughs, and backwater areas can result when the local morphology of the river is altered by placement of dredged materials. Reduction of water flow can change rates of sedimentation and decrease dissolved oxygen levels. Certain aquatic plants important as wildlife foods flourish in a slight current, and reduced current velocities can impair their growth, while adequate oxygen levels are critical to the survival of many species. Disposal on the deeper, open water edges of marshes or in sloughs or backwater areas can reduce depths to a point where additional marshland will develop. Covering of marshland by direct placement of dredged material or through spillover from island disposal can create conditions conducive to the formation of a willow-cottonwood terrestrial habitat.

Most terrestrial disposal sites on the Upper and Middle Mississippi are included in the general definition of open water disposal used in this study. Dredged material placed on open banklines or lightly vegetated shoreline is subject to erosion by wind and water. Wind action can redeposit sand and silt in the river, increasing turbidity and returning dredged materials to the navigation channel from which they were removed. Dredged material placed on many terrestrial sites can also be redistributed during higher stages of the river or during periods of heavy rainfall. Water transport of sediments from a disposal site back to the river can increase local turbidity and may result in relocation of the dredged material to environmentally critical portions of the river such as marshland, sloughs, or backwater areas.

Where dredged materials are placed along beaches or vegetated shoreline, severe damage can be inflicted on local shallow water plant and animal food sources. Shore and marsh birds as well as a variety of mammal species which hunt the shore margins for small fish, insects, crustaceans, and amphibians may be severely impacted. The grasses and marsh plants which line many shore areas are also important food sources for waterfowl. These plants protect some important food organisms from predatory fish, thus, reserving more of the river's production for terrestrial species. Sandy beaches are also important to the reproduction of the turtle population of the Mississippi River. While dredged material disposal areas often create ideal nesting sites, most native turtle species deposit their eggs in late spring or early summer, and disposal on sandy beaches at this time can destroy some part of each year's reproduction.

The vigorous stands of cottonwood or willow found on many alluvial islands and along the channel periphery provide a firm anchor for disposed material. Wind and water transport of dredged material is significantly less from such sites than from lightly vegetated locations. While the ground cover of poison ivy, morning glory, and a variety of vines which forms under stands of willow and cottonwood is smothered by the disposed material, both willow and cottonwood are quite tolerant of stem burial. It is unlikely that disposal to a depth of up to two feet within healthy juvenile stands of either willow or cottonwood and up to three feet within mature stands of these species will kill many trees. Both cottonwood and willow readily form adventitious roots from the main trunk in response to partial burial by dredged material. Other species of the developed river-bottom forest such as oak, maple, and ash are not so tolerant and are easily killed by deep burial of their stems by sand or silt. Many of the original well developed river-bottom forest stands of these hardwood species have been severely reduced by farming of the floodplain. The great productivity of this forest type for wildlife together with its present limited area and sensitivity to burial by dredged material, dictate against disposal in stands of these species.

The St. Paul District has estimated acreages of habitat directly affected by the disposal of dredged material using the records of dredging and disposal operations between 1936 and 1972. Identified disposal sites total 2,370 acres along the Upper Mississippi in the St. Paul District (Figure 1). This represents about one percent of the water surface area of all navigation pools in the District. Habitat changes on the identified disposal areas include the loss of about

388 acres of shallow aquatic habitat and 953 acres of channel border. The aquatic habitat has been converted to about 763 acres of open sandbar, and 578 acres of lowland woods and brush, showing a short term trend for aquatic habitat to be converted to open sand and for open sand to eventually become overgrown with vegetation. About 45 percent of the identified disposal acreage in the District is presently vegetated to varying degrees with trees or brush. Within the remaining 55 percent, some herbaceous vegetation and grasses are present (Corps of Engineers, St. Paul, 1974).

Although the impact of turbidity generated by dredging and disposal operations is not directly within the scope of this study, the effect of dredging on water quality is a primary environmental concern and should be mentioned briefly here. Analysis of dredging and disposal sites on the Middle Mississippi by the Waterways Experiment Station (Solomon et al., 1974) resulted in the observation that channel dredging produces a short-term increase in turbidity in the vicinity of the dredging activity and adjacent areas. Nutrients, toxic metals, and other materials if present in the bottom sediments could be introduced into the water column by the dredging process. On the Middle Mississippi, however, a particle size distribution analysis of sediments collected from the main channel revealed that the bed material consisted primarily of sand, which inherently contains little or no polluted materials. It was concluded that the short-term increase in turbidity from open water disposal of dredged materials would be localized and would not be expected to cause significant problems.

A detailed study was conducted by the St. Paul District during the summer of 1973 to determine the immediate effects of dredging activities on water quality in Pool 8 of the Upper Mississippi River lock and dam system (Figure 44). Sampling stations were established in the immediate vicinity of Crosby Slough and Crosby Island (RM 690.2) where the Dredge Thompson was scheduled to deposit 130,000 cubic yards of material. Benthic samples and water chemistry determinations both upstream and downstream of the disposal site were made prior to, during, and immediately after dredging. A statistical analysis was made of the means of all parameters collected. Significant differences were noted in the means of several parameters, including temperature, turbidity, and dissolved oxygen.

No ecological significance was placed on the differences in the means (before, during, and after dredging) of either temperature or dissolved oxygen. It was concluded that differences in these parameters were more the result of diurnal changes than a function of the dredging process. The turbidity increase was attributed directly to the dredging operations. The greatest contribution to the increase in suspended particles appeared to be the runoff of the dredged material from the disposal site. It was concluded, however, that the effects of dredging on water chemistry at the Crosby Island site were localized, and that significant downstream changes in the chemical parameters measured did not appear to have long term consequences. Dredging creates a local disturbance and the affected water quality parameters return to their pre-dredging status in a relatively short period. The direct physical consequences of the placement of dredged material are of greater magnitude and are essentially irreversible (Corps of Engineers, St. Paul, 1974).

It is of interest that observations at the Crosby Island disposal site confirmed the anticipated redistribution of dredged material from the site by the action of wind and water. Crosby Slough, a small channel just east of the disposal site, has begun to fill with sediments from runoff of the dredged material and is expected to continue to fill from future wind and water erosion. Samples collected before and after dredging indicate that there has been a significant reduction of organisms at the most sensitive locations close to the disposal site. This reduction was due primarily to the overlaying of productive sediments with sand, rendering them unproductive.

Based on limited observations, the impacts of turbidity generated by the dredging process on the riverine environment of the Upper and Middle Mississippi do not appear to be serious. With dredging operations in estuarine or lacustrine locations where bottom sediments may be polluted, the resuspension of polluted materials becomes a dominate factor in the impact of dredging on the environment. The solution of this problem is one of the primary objectives of the Waterways Experiment Station's multi-faceted Dredged Material Research Program (Section 1.2) and so is not considered further here.

The results of a dredged material survey conducted on the Mississippi River between Cairo, Illinois and Hastings, Minnesota in 1969 by the Fish Technical Section of the Upper Mississippi River Conservation Committee provide an excellent overview of those aspects of dredged material disposal of primary environmental concern. Deposition practices were considered harmful when dredged material was deposited as follows:

1. In such a manner as to cause the filling of chutes and side channels.

2. In or near inlets and outlets between the river proper and sloughs or backwater areas.

3. On submerged wing dams and closing structures.

4. At the upstream end of islands.

5. In such a manner that the outwash covers a substantial area of aquatic vegetation in the backwater sloughs and lakes.

6. Without due consideration for established or contemplated public use areas.

In view of limitations existing in some areas for additional deposition without detrimental effects on fish and wildlife, recreation, and navigation, the following general recommendations were made for future disposal sites and other uses of dredged material:

1. Deposit dredged material on existing islands having a low value for timber and wildlife habitat.

2. Consider the construction of sand islands in the wide, flat lower ends of some pools. The principal advantages of such islands would be (1) reduction of wave action, (2) provision of areas for wildlife, (3) better definition of the navigation channel, and (4) creation of additional recreation areas.

3. Develop sand beaches at state, county and municipal parks bordering the river.

4. Provide fill for proposed public access and parking areas sponsored by Federal and State programs.

5. Create dikes in large shallow water areas for waterfowl and aquatic fur animal management.

6. Provide fill in lowland areas of little wildlife value adjacent to communities wishing to have additional space for industrial expansion or other purposes.

5.7.2 <u>Geomorphic and Hydraulic Implications of Open Water Disposal</u>

The preceding summary of the environmental effects of dredged material disposal establishes the impact of disposal on chute channels, sloughs, and backwater areas as a primary environmental concern. The analysis developed in Chapter 3 constitutes the necessary baseline for an examination of the geomorphic and hydraulic implications of open water disposal relative to the evolution of these and other environmentally critical areas.

As a point of departure, evidence gained from hydraulic model studies and observations in the field provide valuable insights into the fate of dredged material disposed at various locations in the riverine environment. For example, the initiation of dredging operations in 1957 for development of the Apalachicola River (Section 5.4.3) aroused considerable controversy within the Mobile District concerning the advisability of within-bank disposal of dredged material as opposed to disposal on the top of the banks. It was agreed that indiscriminate placing of dredged material within banks was not advisable; however, it was also concluded that within-bank disposal would be permissible if it were done at locations where deposition would occur naturally. In a meandering system, these areas of natural deposition generally occur along the downstream portion of a point bar and the upstream portion of a concave bend (Odom, 1966).

A similar controversy in the Portland District concerning over-bank versus within-bank disposal was decided in favor of disposal in the dike fields along the channel periphery (Figure 91). The results of this decision can be seen in the reduced dredging requirements on the Henrici Bar of the Columbia River (Figure 92) and have been discussed relative to the morphologic effects of the lateral redistribution of sediment (Section 5.4.3). In effect, the disposal of dredged material in dike fields is in consonance with the Mobile District's general recommendation of placing dredged material at locations where deposition would occur naturally, since the dike fields create a man-induced depositional environment (Figure 28). Experience with dredged material disposal on the Columbia River led to the conclusion that river sediment is a valuable natural resource (Hyde and Beeman, 1963). Effective use of dredged material depends on the disposal location and the foresight and planning of the engineers dealing with the sedimentation problem. On the Columbia River the use of dredged material for river control and contraction works and for development of recreational areas has provided an economical solution to the disposal problem.

The hydraulic model study of the Manchester Islands, a typical divided reach on the Ohio River, has been used to analyze the impact of alignment on the stability of a dredged cut (Section 5.3.3). Since the Manchester Islands study resulted in recommending continued dredging as the most practical solution to the shoaling problems in the reach, the model was also used to improve dredging procedures by investigating the fate of dredged material disposed at various locations in the divided reach. The results of this investigation provide a basis for determining desirable and undesirable disposal locations in a typical divided reach of an alluvial river. Verification tests of the model, run under existing conditions (Figure 83), revealed that a dredged material disposal site at the head of Island No. 1 used during the 1935 dredging season severely impacted the morphology of the divided reach. In the prototype reach a large bar formed at the foot of Island No. 1 subsequent to the 1935 dredging season. This bar also occurred in the model during verification, and it was noted that the bar was formed almost entirely of material carried downstream from the disposal site at the head of the island. As stage increases both discharge and velocity in the middle channel tend to increase (Tables 14 and 15).

Currents through the middle channel tended to erode the bar at the foot of Island No. 1 and this material together with increased amounts of material moving through the middle channel formed a bar along the left (south) bank of Island No. 2 which extended downstream toward the Kentucky bank (Figure 83). These results confirm the conclusion of the 1969 dredged material survey on the Upper Mississippi that disposal at the upstream end of islands should be avoided.

Following verification and base test runs, a series of runs were made with the Manchester Islands model to determine:

1. The best locations for disposal sites.

2. The areas into which the dredged material might be moved by river currents.

3. The degree of stability to be expected.

The results of these runs are summarized in Table 22 which should be read with reference to Figure 83. Of the possible disposal sites in the reach, the site at the head of Island No. 1 appears to be one of the worst as was indicated by verification test runs. Disposal in the middle channel and along the south bank of Island No. 1 also impact the morphology of the reach and the stability of the navigation channel. Several other possible disposal sites are stable for either high stage or low stage conditions but not for both. The two recommended disposal sites, along the Kentucky bank just above Island No. 1 and along the south bank of Island No. 2 below mile 396, are both in locations that are not subjected to high current velocities at either high or low stage. In addition, both sites are in locations where deposition would tend to occur naturally. While disposal of dredged material along the bankline below the divided reach did not directly influence the reach, dredged material tended to accumulate on the next crossing downstream, and, thus, impacted the system as a whole.

An analysis of the morphology and hydraulics of meandering and divided reaches, with particular emphasis on the changing patterns of high-stage and low-stage flow, can be used as a basis for selecting open water disposal sites that will minimize environmental impacts resulting from redistribution of the dredged material. The most desirable general location is in a region where deposition would occur naturally, such as the downstream portion of a point bar. More specifically, the most desirable disposal sites are in locations that are not directly impacted by high velocities of either high-stage or low-stage flows. The protected depositional environment of the peripheral dike fields along much of the Upper and Middle Mississippi generally meets both of these criteria. However, locations that are desirable based on a hydraulic and geomorphic analysis are not necessarily desirable from a biologic point of view. For example, as desirable as disposal in the dike fields may appear from a physical analysis, disposal in these locations can destroy the prime aquatic habitat created by the rock fill of the dikes. Similarly, the downstream portion of a point bar may offer a stable disposal location for dredged material, but selection of such a site can involve the destruction of the valuable shallow water plant and animal food sources associated with natural sandy beaches or vegetated shoreline.

Equally difficult choices face the river engineer when disposal of dredged material in or near the entrances to chutes, sloughs, and backwater areas is considered. It is generally conceded that direct placement of dredged material in these locations could block the flow of water through the side channels, thereby reducing water flow and consequently water quality. In isolated cases in the past, dredged material has been placed in the entrance of feeder channels for backwater areas; however, it is now Corps of Engineers practice to restrict the disposal of dredged material at the entrances and exits of side channels (Solomon et al., 1974).

Observations on the Upper Mississippi indicate that sloughs and feeder channels to important backwater areas are being blocked by riverborn sediments, but the degree to which dredged material contributes to the problem is not clear. In certain cases, such as the filling of Crosby Slough, observed in conjunction with the study of turbidity effects (Section 5.7.1), the cause and effect relationship is reasonably clear. In others, a combination of factors must be considered. The recent closing of Wyalusing Slough in Pool 10 at RM 627.7 (Figure 44), which feeds a 3600-acre backwater area, can be attributed to a combination of natural sedimentation and dredged material placed in 1968. The Weaver Bottoms-Lost Island area of Pool 5 at RM 745 (Figure 44) has been affected by placement of dredged material and by natural sedimentation to an increasing degree during the last 15 years. Flow of water through the Weaver Bottoms has been virtually stopped by a combination of accumulated islands of dredged material, which line both sides of the navigation channel at the lower end of Weaver Bottoms, and fill or occlusion of the various sloughs which feed this extensive backwater area. (Corps of Engineers, St. Paul, 1974). In the latter two cases it is difficult to assess the impact of disposal practices, as the effects of disposal are superposed on the natural processes of accretion in side channels.

A geomorphic study of the evolution of side channels on the Middle Mississippi River (Section 3.2.3.2) has shown that the ultimate fate of the mature side channel is obliteration by both natural sedimentation and encroaching vegetation. A few large natural side channels on the Middle Mississippi such as Picayune Chute and Santa Fe Chute have remained open for a long period. Picayune Chute (Figures 26 and 27) is in a favorable position at the outside of a bend to receive clear water at its intake. In addition, the large expanse of Devil's Island, has isolated Picayune Chute from sediment-laden high stage flows which might otherwise have produced rapid deposition and closure of the chute. Santa Fe Chute (Figure 25) which is also protected from sediment-laden high stage flows by a large alluvial island has retained its status and has not filled with sediment over the last 90 years (Table 2). Santa Fe Chute is closed by a dike at the inlet and a partial dike at the outlet which may also restrict the movement of sediment into the chute. Except for the few favorably located, larger chute channels, natural and man-made chute channels fill at a rate of up to three feet a year. Backwater channels fill at rates of from one to five inches a year. Evidence from the Middle Mississippi suggests that, unless steps are taken to prevent it, nearly all natural and man-made side channels will ultimately fill with sediment and become indistinguishable from the floodplain.

Thus, the problem of dredged material disposal relative to chutes, sloughs, and backwater areas poses another dilemma for the river engineer. The fill and closure of side channels which alters flow patterns, reduces flow velocity, and accelerates the process of eutrophication, severely impacts the biologic quality of the river system.

However, geomorphic evidence suggests that closure of a chute channel will increase its life by slowing down the natural processes of sedimentation in the chute. Based on observed rates of sedimentation, the life of a chute channel can be increased if it can be isolated on the upper end from the main channel. When isolated, the chute channel becomes a backwater channel in which rates of sedimentation have been observed to be small.

On the Upper Mississippi, consideration is currently being given to dredging openings into backwater areas that have experienced a loss of circulation and reduction in water quality as a result of natural or man-induced deposition in feeder channels. The potential benefits from this use of the dredge as an agent for morphologic change include increased current velocities and improved dissolved oxygen levels, both of which can counter the processes of eutrophication. The complex morphology of the river system dictates that proposals to open back-water areas by dredging be examined in detail for each site. For favorably located side channels such as Picayune Chute (Figure 26), the procedure could have long range beneficial effects, however, the potential for adverse impacts as a result of the process also exists. Converting a closed slough into an open chute channel could greatly accelerate sedimentation, particularly if the opening is oriented to induce the flow of sediment-laden high stage flows through the opened channel. The introduction of sandy, main channel sediments which would settle on the organic-silt backwater bottom sediments could also produce a decrease in biologic productivity. In 1974, the St. Paul District opened a 55-foot long by 6-foot deep dredged channel at Mule Bend

(RM 748.4) just above the Weaver Bottoms in Pool 5. The effects of this experiment are currently being monitored.

The 1969 dredged material survey on the Upper Mississippi suggested the limited use of alluvial islands for disposal of dredged material. The recommendation was made in view of the limitations existing in some areas for continued disposal of dredged material without detrimental effects on fish and wildlife, recreation, and navigation, and included only islands having low value for timber and wildlife. The geomorphic processes of island building (Section 3.2.3.2) which produce the characteristic crab claw shape of many alluvial islands on the Upper Mississippi lend support to this recommendation. The peripheral natural levee that results from the island building process (see for example the Crider Islands, Figure 46) would provide confinement and protection for dredged material disposed in the low-lying, boggy, island interior during all but the highest stages of the river. Physically, then, the island interior provides an ideal disposal site for dredged material, but ecologically, utilization of an alluvial island for disposal may not be acceptable. One of the primary impacts would be the effect of stem burial on the hardwood species of the mature river-bottom forest. Stands of oak, maple, ash, and hickory present in the island interiors would be severely impacted by stem burial, resulting in the loss of this highly productive forest type. Again, the physical benefits of island disposal must be weighed against the direct ecological disadvantages.

The effects of disposal in the terrestrial habitat of an alluvial island depend primarily on the type of vegetation present. Stands of willow and cottonwood can survive stem burial and will recolonize a sandy disposal site. Even stands of the hardwoods can benefit to a degree from disposal. Thinning a mature hardwood stand having only a sparse understory of native seedlings can create open areas where regrowth of seedlings can occur once the dredged sand is covered by a layer of silt from flooding (Corps of Engineers, Rock Island, 1974).

Creating open spaces in an island interior can also enhance its potential for recreation. The potential benefits to man from the selective disposal of dredged material on and adjacent to an alluvial island are outlined in the Rock Island District's Environmental Impact Statement (1974). Bass Island, at RM 448 in Pool 17, provides an example of the selected use of dredged material for the recreational enhancement of an alluvial island. Bass Island (Figure 108) has features which are typical of many alluvial islands on the developed river, including contraction dikes, an existing disposal area along the main channel bank, and a heavily wooded island interior. Potential features of the island (Figure 109) include general recreation areas along the main channel side of the island and more primitive areas toward the interior and eastern shore, all developed by selective disposal of dredged material. Trails have been shown on the potential use sketch to tie together walk-in camp sites and unique natural features such as marshland, inland ponds, and the woodland community.

When the environmental impacts of dredged material disposal are compared with the geomorphic and hydraulic implications of open water disposal, areas of serious conflict become apparent. In general, locations that constitute acceptable disposal sites based on an analysis of the physical processes of the river system are unacceptable when the biological processes are considered. Where compromise solutions cannot be found, alternatives to the current practice of open water disposal of dredged material must be sought. A summary of possible alternatives is presented in Figure 110. These alternatives are discussed in detail in the various environmental impact statements for operation and maintenance of the 9-foot channel. In addition the Waterways Experiment Station's Dredged Material Research Program (Section 1.2) includes detailed analyses of many aspects of erosion control, dredged material placement, dredge operations, and beneficial uses of dredged material. Accordingly, these alternatives will not be examined in detail in this study. It should be noted, however, that many of these alternatives require either major modifications or additions to the existing dredging plant, or major changes in current dredging operations. For example, remote disposal, central disposal, or removal of dredged material from the floodplain are generally not within the current capabilities of hydraulic dredges using discharge pipeline. These alternatives would require the use of innovative techniques such as a shuttle barge system for dredged material disposal. An alternative based on the geomorphic and hydraulic characteristics of the river system, which avoids the direct biologic impacts of bankline and island disposal, and at the same time is generally within the capabilities of existing dredge plant, is investigated in the following section.

5.7.3 Disposal of Dredged Material in the Thalweg--An Alternative

A comparison of the environmental impacts of dredged material disposal with the geomorphic and hydraulic implications of open water disposal has revealed areas of serious conflict. In general, locations that constitute the most desirable disposal sites based on an analysis of the physical processes of the river system are undesirable when the biological processes are considered. Physically, the best locations for disposal are in regions where deposition would occur naturally. These include the downstream portion of point bars and other locations not directly subject to high velocities during either high-stage or low-stage flow. The man-induced depositional environment of the dike fields offers protected disposal sites, as does the interior of many alluvial islands. However, disposal in these locations usually involves serious and often unacceptable environmental impacts. The only remaining significant portion of the riverine environment that offers potential disposal locations is the main channel or thalweg region of the river itself.

The concept of disposal of dredged material in the main channel is not without precedent. With reference to dredging on the Columbia River, Hopman (1972) notes that repetitive shoaling and the subsequent dredging it necessitates occurs primarily at specific locations. As a result, the availability of disposal sites in these critical areas becomes limited due to past use of the most desirable areas and physical development of the remaining areas. On the Columbia, the decrease in available disposal sites and the continuing requirement for navigation channel maintenance demanded a second look at the total dredging operation. The solution suggested by Hopman for the Columbia River was a "cut and fill" approach to dredging. With this approach, the crest of a shoal or crossing which requires deepening is moved downstream into the nearest trough or pool, much as a highway engineer removes earth from its original position in a cut and deposits it in the nearest fill.

The cut and fill approach was one of several dredging alternatives tested in 1971 by the Portland District. In an attempt to improve maintenance of small isolated shoals with increased efficiency and reduced costs, one of the District's small tugs was fitted to pull a standard agricultural-type harrow over a selected shoal, an operation reminiscent of early dredging methods on the Mississippi (Section 5.2.1). The test was conducted over a two-month period in July and August 1971, and is estimated to have successfully removed about 50,000 cy of material at a cost per yard well below that of hydraulic pipeline dredging. Additional benefits included keeping the dredged material near the bottom which reduced turbidity and suspended sediment problems, and avoiding the impacts of wetland or shallow water disposal (Corps of Engineers, Portland District, 1973).

The concept of thalweg disposal can also be supported from a general geomorphic point of view. Although the longitudinal profile of a river can be conceptually described by a decreasing exponential relationship such as the Shulits equation (6) and visualized as a smooth, concave-upward curve (Section 3.2.1.3), the detailed longitudinal profile of a river is more complex. Detailed profiles along the Upper Mississippi (Figures 44 and 51) and the Middle Mississippi (Figure 62) appear as an irregular series of high points and low points. In a meandering thalweg stream the high points of the profile generally correlate with the crossings and the low points with the deep bendway pools. At high stage, sediment tends to flush from the pools and adjacent point bar areas and accumulate on the crossings, reducing the depth of flow. At low stage, the process is reversed; however, low stage scour on the crossings is often not sufficient to produce required navigation depths during the low-water season. This sequence of deposition and scour, which is described in more detail in Section 3.2.2.3, results in repetitive dredging requirements on the crossings.

In effect, the pools and crossings of an alluvial river alternately serve as sediment source and sink areas as the sediment is transported downstream. In regard to maintenance of a navigation channel the crossings can be visualized as sediment source areas and the pools as sediment sink areas. The concept of thalweg disposal, then, involves dredging a crossing source area and disposing the dredged material in a downstream pool or sink area.

Anderson (1975) recommended in a recent proposal to the Dredging Requirements Work Group of the Great River Environmental Action Team (GREAT) an investigation of the feasibility of riverine disposal of dredged material using a physical hydraulic model. In this proposal Anderson noted that the process of transporting dredged material from a crossing to the succeeding downstream pool is equivalent to adding sufficient energy to increase the local transport rate at the crossing. The use of a physical model was proposed to determine the limiting ratio of annual dredging to annual total transport. If the volume to be dredged were small compared to the total annual transport, the small energy increment required to move material from a crossing to a downstream pool should have little influence on the regime of the river. However, if the volume dredged were of the same order of magnitude as the annual volume transported, significant geomorphic change may occur.

Based on the results of testing a "cut and fill" approach to dredging in the field, and on an analysis of the morphology of the crossing and pool sequence of a meandering thalweg river, the concept of thalweg disposal warrants further investigation. The potential environmental benefits are numerous, particularly in regard to avoiding shallow water or wetland disposal with the consequent impacts on chute channels, sloughs, and backwater areas. As the main channel region of the Upper and Middle Mississippi is generally biologically rather sterile (Section 5.7.1), the direct biologic impacts of thalweg disposal would be minimal. Based on limited observations, the impacts of turbidity resulting from current dredging operations on the Mississippi do not appear to be serious (Section 5.7.1), This conclusion would also pertain to the process of thalweg disposal of the relatively clean, sandy sediments of the Upper and Middle Mississippi main channel. Settling velocities of the sediments dredged from the main channel should generally be sufficiently high to limit the downstream influence of turbidity generated by thalweg disposal.

Because of the apparent feasibility and potential benefits of thalweg disposal, a mathematical model developed to assess future geomorphic changes in Pools 24, 25, and 26 on the Upper Mississippi was adapted to permit an evaluation of the geomorphic and hydraulic impacts of dredging material from a crossing and disposing it in a downstream pool. The model is based on a mathematical model for water and sediment routing developed by Y. H. Chen (1973), and was adapted by Chen and Lagasse to investigate the feasibility of thalweg disposal. The following brief summary of the characteristics, capabilities, and limitations of the mathematical model is extracted from Chen (1973) and Appendix B to "A Geomorphic Study of Pools 24, 25, and 26 in the Upper Mississippi and Lower Illinois Rivers" by Simons, Schumm, Stevens, Chen, and Lagasse (1975).

The model is developed by describing the unsteady flow of sediment-laden water with the one-dimensional partial differential equations representing the conservation of mass for sediment, and the conservation of mass and momentum for sediment-laden water. The effects of locks and dams in the detailed study area, and the interactions between the Mississippi River and its main tributaries, as well as the influence of dredging operations between 1949 and 1970 are considered in the modeling.

The one-dimensional differential equations of gradually varied, unsteady flow in natural alluvial channels can be derived based on the following assumptions:

1. The channel is sufficiently straight and uniform in the reach so that the flow characteristics may be physically represented by a one-dimensional model.
2. Hydrostatic pressure prevails at every point in the channel.
3. The water surface slope is small.
4. The density of the sediment-laden water is constant over the cross section.
5. The resistance coefficient is assumed to be the same as that for steady flow in alluvial channels and can be approximated from resistance equations applicable to alluvial channels or from field data.

The three basic equations thus derived are:

1. The sediment continuity equation

$$\frac{\partial Q_s}{\partial x} + p\,\frac{\partial A_d}{\partial t} + \frac{\partial AC_s}{\partial t} - q_s = 0 \qquad (46)$$

2. The flow continuity equation

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + \frac{\partial A_d}{\partial t} - q_\ell = 0 \qquad (47)$$

3. The flow momentum equation

$$\frac{\partial \rho Q}{\partial t} + \frac{\partial \beta \rho Q V}{\partial x} + gA\,\frac{\partial \rho y}{\partial x} = \rho g A\,(S_o - S_f + D_\ell)$$

or

$$\frac{\partial \rho Q}{\partial t} + V\,\frac{\partial \beta \rho Q}{\partial x} + \beta \rho V\,\frac{\partial Q}{\partial x} - \beta \rho V^2 T\,\frac{\partial y}{\partial x} + gA\,\frac{\partial \rho y}{\partial x} = \rho g A (S_o - S_f + D_\ell) + \beta \rho V^2 A_x^y \qquad (48)$$

in which x = the horizontal distance along the channel; t = time; Q_s = the sediment discharge; p = the volume of sediment in a unit volume of bed layer given ρ_b/ρ_s; ρ_b = the bulk density of the sediment forming the bed; ρ_s = the density of sediment; A_d = the volume of sediment deposited on the channel bed per unit length of channel, the value of which is negative when bed erosion occurs; A = the water cross-sectional area; C_s = the mean sediment concentration on a volume basis given by (Q_s/Q); Q = the flow discharge; q_s = the lateral sediment inflow per unit length of channel; q_ℓ = the lateral inflow per unit length of channel; ρ = the density of sediment-laden water given by $\rho_w + C_s(\rho_s - \rho_w)$; ρ_w = the density of water; β = the momentum coefficient; V = the mean flow velocity; y = the flow depth; g = the gravity acceleration; S_o = the bed slope; S_f = the friction slope; D_ℓ = the dynamic contribution of the lateral discharge given by $(q_\ell V_\ell / Ag)$; V_ℓ = the velocity component of lateral inflow in the main flow direction, and A_x^y = the departure from a prismatic channel given by $(\partial A/\partial x)_y$ (Figure 111).

The three equations contain three basic unknowns Q, y and A_d. The other variables in the equations must be expressed as a function of these three unknowns in order to obtain a solution. These functions are given by the following supplementary relationships or assumptions:

1. The geometric properties of cross sections are expressed as a function of stage from the known channel geometry.

2. The mean bed slope

$$S_o = - \partial z/\partial x \tag{49}$$

in which the initial bed elevation is known and its change is related to the variable A_d.

3. The friction slope S_f is a function of flow and channel characteristics. The resistance functions such as Manning's or Chezy's equations can be employed to relate S_f to the basic unknowns.

4. The lateral inflow q_ℓ consists of two components, $q_{\ell 1}$ and $q_{\ell 2}$, induced by natural and man-made activities, respectively. For overbank flow, the natural-induced lateral inflow is related to the change of water surface elevation Δh over a time period Δt by the following expression:

$$q_{\ell 1} = - \frac{A_f}{\Delta x \Delta t} \Delta h \tag{50}$$

where A_f is the surface area of the floodplain, and Δx is length of the floodplain along the main channel. Equation (50) is based on the assumption that the transverse water surface (normal to the main flow direction) is horizontal. Accordingly, the lateral sediment inflow q_s has its natural and man-induced components, q_{s1} and q_{s2}, in which

$$q_{s1} = q_{\ell 1} C_b \tag{51}$$

and C_b is the sediment concentration at or near the riverbank.

5. The sediment discharge can be estimated from field surveys and/or from the available theories.

To account for the effects of locks and dams the continuity equations for both sediment and water discharge are applied across each lock and dam and supplemented by a gate discharge equation (Figure 112). The interaction between the Upper Mississippi and each of its major tributaries in the study area (the Illinois and Missouri Rivers) is simulated by continuity and energy relationships written at each tributary confluence (Figure 112). The set of basic equations and supplementary relationships (46-51) plus the equations written across each lock and dam and tributary confluence govern the water flow and sediment movement in the study reach. Changes in flow and channel characteristics can be assessed from the solution of these equations. Because of the nonlinearity of these equations, the only feasible solution is by numerical methods.

The set of equations described above can be solved by a linear-implicit method using a digital computer. The finite-difference approximations employed to express the values and the partial derivatives of a function f within a four-point grid formed by the intersections of the spacelines x_i and x_{i+1} with the time lines t^j and t^{j+1} are given by Figure 113.

$$f \approx \frac{1}{2}\left(f_i^j + f_{i+1}^j\right) \tag{52}$$

$$\frac{\partial f}{\partial x} \approx \frac{1}{\Delta x}\left(f_{i+1}^{j+1} - f_i^{j+1}\right) \tag{53}$$

and

$$\frac{\partial f}{\partial t} \approx \frac{1}{2\Delta t}\left[\left(f_i^{j+1} - f_i^j\right) + \left(f_{i+1}^{j+1} - f_{i+1}^j\right)\right] \tag{54}$$

in which f represents Q, A, y, etc. All the variables are known at all nodes of the network on the time line t^j. The unknown values of the variables on the time line t^{j+1} can be found by solving the system of linear algebraic equations formulated by substitution of the finite-difference approximations (52), (53), and (54) into the basic set of equations as well as the equations at tributary confluences and locks and dams.

This results, in this case, in a system of linear algebraic equations which constitutes a diagonally dominant matrix. Any of the standard methods, such as Gaussian elimination or matrix inversion, can be used for its solution; however, a double sweep method was applied by Chen for solving this system of linear equations. This method offers the following advantages: (1) the computations do not involve any of the many zero elements in the coefficient matrix; this saves considerable computing time; and (2) the required computer core storage is reduced significantly from that required by other methods.

Calibration of a mathematical model involves evaluation and modification of the supplementary relations to the basic equations from field data and/or theories such that the mathematical model will reproduce the historical response of the modeled river system. This is similar to calibration of a physical model. To perform the mathematical model calibration, the following information is required:

(a) hydrographic maps of the modeled river reach,

(b) hydrographs of stage, flow and sediment discharge, and

(c) geological and physical properties of the bed and bed material.

From (a), one can evaluate the geometric properties of the river reach. The relations for S_f, Q_s, q_ℓ and V_ℓ can then be evaluated from (b) and (c). If part of data is not available, relations based on experimental, empirical, or theoretical approaches can be used. However, calculated results are only as good as the calibration relations. More specifically, the resistance function for S_f and the sediment transport function for Q_s must be tested and modified to accomplish the model calibration, that is, until the historical data along the river reach can be reproduced by the mathematical model.

The study reach (Figure 112) was divided into 75 sections with space increments ranging from 0.4 miles at a lock and dam to 20 miles in the Missouri River. By utilizing data listed under a, b, and c above, the supplementary relations to the three basic equations were evaluated at all 75 sections in the modeled river. Calibration was intended to reproduce the flow characteristics and geomorphic changes of the study reach from 1939 to 1971 when routing the 1939-1971 discharge hydrograph through the modeled river. Numerous trials were required to modify the resistance function and the sediment transport function in order to reproduce the known historical changes. As an example of model calibration, the 1965 water surface profiles in the Pool 24, 25, and 26 reach are compared with measured stages in Figure 114. Excellent agreement between the measured and calculated values is apparent.

The limitations of the mathematical model relate primarily to its one-dimensional character. Such a model is quite effective in studying the short-term and long-term river responses to development in a long river reach. However, when the space increments are chosen to be relatively large to operate the mathematical model efficiently, and sediment is assumed to be uniformly distributed over the channel width, only the general pattern of the river geomorphology can be considered. To study a special reach of river in detail, either this river reach is subdivided into a large number of segments to apply the mathematical model or a combination of physical model and mathematical model might be utilized. For the study of a specific problem such as dredging and disposal, the one-dimensional character of the model requires that material removed from the crossing and disposed in the pool be distributed evenly over the cross section in each case.

With these limitations in mind, the mathematical model as calibrated for the Pool 24, 25, and 26 detailed study area can be used to study the effects of dredging a crossing and disposing the dredged material in a downstream pool. The RM 269 to RM 264 reach of Pool 25 constitutes the most serious dredging problem area in the three pools (Figure 107). Accordingly, one crossing in this reach that has required extensive dredging and the pool and crossing immediately downstream were identified and modeled by adding 18 additional sections between RM 269.0 and RM 265.0. This reduced the distance between adjacent sections to as little as 0.8 of a mile.

A dredged cut 3 ft deep and 950 ft long (from RM 268.91 to RM 268.72) was made in the crossing area over the channel width by assigning a suitable value to q_{s2}, the man-induced lateral sediment flow (Equation 51). This resulted in approximately 200,000 cy of dredged material which was disposed of in the downstream pool area (RM 268.46 to RM 268.28). The cut was made at the beginning of the low-water season and riverbed level changes in the modeled reach were computed during the next year for the 2-year annual hydrograph. Results are shown in Figure 115.

The initial bed profile with dredged cut and disposal site is shown in Figure 115a. The dark vertical bars represent the upstream and downstream locks and dams. The discharge hydrograph, number of months after dredging, and discharge volume in thousands of cfs are shown at the top of each figure. After a 4.4 month low-flow period, the bed level showed only small changes (Figure 115b). With the flood entering the reach (Figure 115c), the dredged cut was filled in rapidly and the bar at the disposal site was moved to a downstream crossing as shown in Figure 115c. After one year, both crossing and pool returned to nearly the natural state. This sequence coincides with the influence of stage outlined in Section 5.3.4 and shown graphically in Figure 88.

The sequence is illustrated in more detail in Figure 116 which shows the riverbed level changes with time in the crossing and the pool areas. Riverbed levels are compared with those that would occur during the same year if no dredging had been performed. Without dredging, the crossing aggrades and the pool area degrades. Following dredging from the crossing and disposal in the downstream pool, the dredged cut persists for about 180 days and then begins to fill. The disposed material in the pool also remains stable for about 180 days, but then begins to scour. At one year after dredging the bed level changes are approximately equal to those without dredging. With a normal May through October dredging season on the Upper Mississippi this dredged cut in an extremely troublesome reach could be expected to remain open through the dredging season under the flow conditions simulated. For the location and flow conditions tested, the accretion on the next crossing downstream did not exceed 1.5 feet.

To test the impact of the quantity of material disposed in the thalweg of the downstream pool, model runs were made under the same conditions as described above, but with, first, all the material disposed on the adjacent floodplain, and, second, half disposed on the floodplain and half in the downstream pool. Figure 117 shows the general location of the dredging and disposal operation and the impacts of each alternative. With all the dredged material disposed on the floodplain (Figure 117b), the downstream pool scoured much more during the year following dredging than if no dredging had been done. The sediment derived from this scour may be deposited, in part, on a downstream crossing. Disposing of half the dredged material in the downstream pool (Figure 117c) reduced the amount of scour in the pool area and, thus, presumably also reduced the amount of material available for deposition on the next crossing downstream. The riverbed profile one year after disposal of half the dredged material in the downstream pool is quite similar to the profile resulting from disposal of all the dredged material in the downstream pool (Figure 117c). It should be noted that very little accretion occurs on the downstream crossing for the average conditions represented by the 2-year annual hydrograph.

To investigate the impact of thalweg disposal under a different flow condition, the study area was subjected to a variety of hydrographs including a one-year annual hydrograph. The results of subjecting the model to an annual hydrograph with small flow volume and reduced peak are shown in Figure 118. With all the dredged material disposed on the floodplain (Figure 118b), the dredged cut remains essentially unchanged during the year following dredging, and the profile through the downstream pool and crossing coincides closely with that expected under natural conditions. However, with disposal of all the dredged material in the downstream pool under conditions of a small annual hydrograph, the next crossing downstream is strongly impacted. Figure 118c shows accretion to almost 2.5 feet over natural river conditions at the downstream crossing. With disposal of half the dredged material in the pool, accretion on the downstream crossing is reduced by about 1.0 feet (Figure 118c). There is a risk, then, that if the annual hydrograph which follows dredging and thalweg disposal is small, the material disposed in the pool area will be scoured and will deposit on the next crossing downstream. The result could be inadequate navigation depths and consequent dredging requirements on that crossing. This is particularly true if the crossing below the disposal pool is already experiencing dredging problems, and could also be true if a divided reach or unstable straight reach is located below the disposal pool.

The model was also subjected to an annual hydrograph with a larger flow volume and a higher peak than the 2-year hydrograph to evaluate the impact of an extreme event on the thalweg disposal process. The 1973 flood hydrograph with a recurrence interval of 50 years was used to represent an extreme event. Neither the dredged cut on the crossing nor the disposal bar in the pool persisted for long under the high sediment transport conditions of a flood of this magnitude. After 100 days of the hydrograph, bed elevation change closely approximated that to be expected under natural river conditions with no dredging. The potential for a short-term critical period of accretion on the crossing below the disposal pool was noted at about 41 days into the hydrograph.

A geomorphic analysis of the crossing and pool sequence supported by mathematical modeling of a particularly troublesome reach of Pool 25 on the Upper Mississippi indicates that dredging from a crossing and disposing the dredged material in a downstream pool does constitute a feasible alternative to the disposal problem. The process involves a degree of risk of impacting the integrity of the navigation channel downstream from the pool, particularly if dredging is followed by a small discharge hydrograph. However, the risks incurred would be outweighed by the potential environmental benefits at many locations. Based on data currently available, the direct biologic impacts of disposal in the main channel as well as possible secondary impacts from turbidity generated by the disposal process appear minimal. In addition, the serious ecological problems associated with open water disposal on marshlands and near chute channels, sloughs, and backwater areas are avoided by the process of thalweg disposal. Although conditions downstream of a proposed disposal site may preclude thalweg disposal at certain locations, in many cases disposing only a portion of the dredged material along the thalweg would still result in reduced environmental impacts. Consequently, the concept of thalweg disposal offers a viable alternative to both long term and emergency disposal requirements. The results of the limited analysis of this study are sufficiently promising to warrant additional investigation.

5.8 Summary

Contraction works and flow regulation by locks and dams are generally considered permanent solutions to the problem of providing navigable depths in a river system. In comparison, dredging must be classed as a temporary solution to the problem of maintaining a navigation channel. Dredging and other temporary solutions to the navigation problem have several distinct advantages over more permanent solutions.

Temporary solutions are relatively simple and direct in their application. They also afford a degree of flexibility in meeting unexpected or changing requirements in a waterway system not possessed by more permanent solutions which require large capital investment and long range planning. Temporary solutions, however, all attempt to change the local configuration of the channel without changing the forces that have produced that configuration, and for this reason can be expected to prevail against the dominant forces of the system for only a limited period of time. Field experience has shown that, although temporary, dredging is usually a necessary adjunct to programs for river system development by more permanent means.

Although maintenance of navigation depths on the Mississippi has been attempted with a wide variety of dredging devices, it was only after the development of the hydraulic pipeline dredge in the 1890's that dredging assumed an important role in navigation channel maintenance. Both the cutterhead dredge and dustpan dredge are currently employed in the study area. Each has a different mode of operations and different capabilities, however, each is well suited to navigation channel maintenance. Both the cutterhead and dustpan dredge discharge dredged material as a water-sediment slurry through a discharge pipeline; but, because of its slower rate of advance, the cutterhead dredge is better adapted to discharging overbank through shore pipe. In addition, the cutterhead dredge is better adapted to development dredging operations, such as the cutting of a pilot channel for a meander loop cutoff.

Dredging operations on the Mississippi River are closely related to the annual cycle of high and low flow. Channel condition soundings are usually not initiated until after partial recession of the annual spring high water. Initial soundings are followed by detailed surveys which lead to a determination of dredging requirements for the coming season. The dredging operation at a particular site is based on the detailed survey which includes size and alignment of the cut as well as disposal area locations. The stability of the dredged cut depends to a great extent on these three factors. Dredging operations normally commence around the first of May and continue as late as November, depending on the weather.

The dredging of a channel through a crossing or shoal can be considered successful if the cut achieves the required navigation depth with a minimum of excavation, and if the cut does not require re-dredging during the navigation season. The depth, width, and alignment of the cut all influence its stability, and, thus, the degree of success achieved. Some indication of the effect of depth and width on the stability of a dredged cut can be obtained from a relatively simple hydraulic analysis. The influence of alignment on stability is best approached through field experience or a hydraulic model study.

Combining the Darcy-Weisbach equation in an open channel flow form with the continuity equation and the definition of shear velocity indicates that bed shear, and thus, contact-load transport capacity, varies inversely as the square of the depth, for a channel with constant width, roughness, and discharge. This simple analysis reveals that the policy of advanced maintenance dredging or overdepth dredging at constant width as currently practiced on many rivers in the United States can significantly decrease the transport capacity of the dredged cut and thus its stability. This conclusion is supported by an analysis based on a replotting of the Colby sediment transport curves into a form suitable for investigating the effect of variation in depth at constant width on transport capacity. In a specific case, a 30 percent increase in depth at constant width can decrease transport capacity by as much as 80 percent. A policy of overdepth dredging can be supported only if the economic benefits that result from the availability of greater depths for longer periods and from more efficient scheduling of dredging operations balance the added cost and the impact of disposing greater quantities of dredged material.

While a constant width analysis sheds some light on the stability problems associated with overdepth dredging, in the general case the influence of width and depth changing concurrently must be considered. As width and depth of a channel or dredged cut change, flow area and wetted perimeter also change. Consequently, the influence of width and depth on flow velocity, and thus on bed shear and transport, can be established through the hydraulic radius which represents the ratio of area to wetted perimeter. Assuming that slope and resistance are constant, velocity can be directly related to the square root of the hydraulic radius through the Chezy equation. In this manner the impact of dredging on the hydraulic radius of a section can be used as a criteria for stability. If the dredging process adds to the wetted perimeter faster than it increases flow area, then the hydraulic radius and velocity decrease, indicating a tendency toward shoaling in a dredged cut. Conversely, if area increases faster than wetted perimeter, hydraulic radius and velocity increase, indicating possible scour in the dredged cut or a tendency toward stability. From this simple analysis design curves can be prepared to permit choosing values of width and depth which will produce this tendency toward stability in the cut.

Although a steady, uniform flow analysis indicates that under optimum conditions a dredged cut can be designed to be stable, the influence of both alignment and unsteady flow on stability must also be considered. The necessity of considering these factors is emphasized by test dredging programs which reveal that dredged cuts can be extremely unstable under conditions of nonuniform, unsteady flow in a natural river. A hydraulic model study of the Manchester Islands reach of the Ohio River provides the opportunity to evaluate the influence of alignment on dredged cut stability. In this study the effects of a variety of dredged channel alignments coupled with closure of chute channels were investigated in a typical divided reach of the Ohio River. This study highlights the difficulty of aligning the dredged cut with both the high-stage and low-stage flow patterns and underscores the consequences of mal-alignment of a dredged channel. Under two plans tested, the dredged cut was oriented properly for low-stage flows, but high-stage flows produced crosscurrents which resulted in shoaling and obliteration of the head of the dredged cut. Under the one test plan that was configured to force both low-stage and high-stage flows into the dredged channel, the channel remained stable. In this case, however, high velocities in the dredged channel placed so much sediment in motion that serious deposition problems resulted at the downstream end of the channel. An additional significant result of this study was the conclusion that the secondary effects associated with permanent solutions to the shoaling problem were unacceptable. Continued maintenance dredging with emphasis on improving dredging procedures was recommended.

The impact of unsteady flow on the stability of a dredged cut can be evaluated by an examination of the sensitivity of a dredged cut to changing stage. Data to support this analysis was obtained by the Memphis District during the 1930 and 1931 dredging seasons. Comparing river stage, average depth over crossings, and dredging requirements shows that, in general, crossings are built up and dredged cuts filled when river stage falls after a period of high flow. During a period of falling stage the available depth on crossings decreases rapidly unless dredging is initiated to counteract the trend. With rapidly falling stages, dredging requirements to maintain navigation depths on the crossings increase rapidly. During an extended period of low flow, dredging requirements taper off, but increase again if the stage rises or fluctuates.

Thus, the unsteady character of natural alluvial channel flow has a major impact on dredged cut stability. This impact is apparent in the changing patterns of erosion and deposition in the crossing and pool sequence which accompany variations in stage. It is also evident in the significantly different flow patterns of the high-stage and low-stage river. Although it may be possible to design a stable dredged channel for uniform, steady flow conditions, the changing conditions of flow on a natural river lead to the conclusion that a dredged cut must be considered successful if it survives a single navigation season.

Dredging operations in the riverine environment are generally maintenance oriented. However, the dredge does provide the river engineer with a means of rapidly altering channel configuration and accelerating morphologic processes in support of river development programs. In this regard, dredging can be considered a morphologic

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agent responsive to engineering requirements. It was in this context that dredging was applied to the Arkansas River development project. Here dredging was used to accelerate the expected natural degradation below newly constructed navigation locks and dams, and to break up the formation of an armor layer of coarser particles which would retard natural degradation. On the Lower Mississippi dredging has been employed to radically alter river morphology in support of the river stabilization program. The dredging of cutoff channels in a meandering river system greatly accelerates the natural processes of meander loop cutoff and produces significant changes in river characteristics. Artificial cutoffs which are intended to produce a gently sinuous river, reduce flood heights, and improve navigation conditions can also increase water surface slope, flow velocity, and, in general, may upset the equilibrium of the river.

The application of hydraulic and mechanical dredges to obtain gravel and other materials from a river for construction and related uses is neither development nor maintenance oriented; but gravel mining with dredges can seriously impact river morphology and stability by removing significant quantities of the coarser sediments from a river. This coarser fraction, particularly gravel, has a tendency, through hydraulic sorting, to armor the riverbed, thereby retarding or arresting excessive scour, stabilizing banks and bars, and preventing excessive sediment movement. Gravel armored sandbars serve as semipermanent controls that define river form. Removal of the gravel armor from such features can lead to erosion and loss of this control. On the Lower Mississippi, gravel mining has been related to a decrease in average particle size in many reaches, and has been associated with the development of divided reaches in many previously stable locations. There is no evidence to support a conclusion that gravel mining is having an equal impact on the morphology of the Upper Mississippi. However, projected increases in the use of sand and gravel resources in the Upper Mississippi basin do suggest that gravel mining will eventually impact the morphology of the upper river.

Because of the relative instability of a dredged channel in an alluvial river, it must be concluded that the dredged cut itself does not significantly alter river morphology. For this reason, dredging is generally coupled with the use of contraction works, such as dikes and revetment, to provide a navigation channel. The combined use of dredging, dikes, and disposal of dredged material in the dike fields can induce major changes in the cross-sectional characteristics of a river. The lateral redistribution of sediments from the main channel to dike fields on the channel periphery has produced a marked decrease in the width to depth ratio in many sections of the study area.

Channel shape in the form of the width to depth ratio has been related by many researchers to the capacity of a channel for the transport of both suspended and contact load. The best hydraulic section for the transport of contact load is a wide shallow section (large width to depth ratio), and conversely a narrower, deeper section is more efficient for the transport of suspended load (small width to depth ratio). To the extent that dredging and contraction reduce the width to depth ratio at a section, they increase the channel's capacity for transport of the suspended portion of the bed-material load but decrease the capacity for transport of contact load. Thus, the lateral redistribution of sediment by dredging, when combined with contraction works, also contributes to morphologic change in a river system.

A comparison of geomorphic data with records of dredging quantities required to obtain and maintain the 9-foot channel on the Mississippi provides a basis for determining those factors that influence dredging requirements on an alluvial river. Such non-engineering factors as project funding and dredge plant availability can influence the records of dredged volumes and limit, to a degree, the establishing of a direct cause and effect relation between dredged quantities and hydraulic or geomorphic change. Nevertheless, increasing dredged volumes in the detailed study area on the Upper Mississippi appear closely related to such factors as:

1. Initial dredging requirements to make the transition from the 6-foot channel to the 9-foot channel.

2. Extended periods of abnormally low flow where lack of water in the system becomes a controlling factor.

3. Extended periods of unusually high flow.

4. Operational policies such as the practice of overdepth and overwidth dredging.

5. Effectiveness and efficiency of dredging operations.

In contrast, on the Middle Mississippi were relationships under conditions of open river regulation are much less complex, a tendency toward decreased dredged volumes can be directly related to contraction efforts. In fact, dredging requirements in a short study reach of the Middle Mississippi have been completely eliminated by carrying contraction efforts to an extreme. Contraction efforts on the Middle Mississippi have, however, been partially responsible for a significant increase in river stage for higher discharges. Increased contraction to minimize dredging can be applied only so long as secondary effects in terms of hydraulic and geomorphic change, are acceptable. Beyond this point, continued maintenance dredging must be accepted.

Plots of dredging frequency and dredged volume by location provide a means of relating dredging requirements and geomorphic characteristics. Dredging trouble spots, that is, areas of high dredging frequency or large dredged volume in the detailed study area, can be closely related to the following factors:

1. Location of locks and dams.
2. Dredging location relative to pool primary control point.
3. Straight reaches.
4. Divided reaches.

Combinations of these factors produce particularly troublesome reaches. In the detailed study area the most serious dredging problems occur in straight reaches which are located above the pool primary control point, and which are also divided by alluvial islands. Surprisingly, in the reach studied, dredging requirements did not correlate closely with tributary locations, however, this is a factor in other areas.

The application of these geomorphic indicators of dredging problem areas to a typical field problem illustrates their problem solving potential and value to the river engineer. The problem selected involved the identification of dredging sites for a pilot program to assess the effects of reduced overdepth dredging on the navigation channel in the St. Paul District. Geomorphic indicators were used to identify those locations which would have the greatest probability of success in reducing the depth of maintenance dredging without incurring an undue risk of channel failure. Dredging frequency and channel condition surveys after dredging were used to test the validity of the analysis when applied to 10 representative locations on the Upper Mississippi. The geomorphic analysis resulted in the selection of four of the five locations with a history of low dredging frequency and did not select any of the high dredging frequency locations for inclusion in the pilot program. Based on this case study, it can be concluded that an analysis of the hydraulic and geomorphic response of the river to dredging provides the river engineer with a methodology applicable to the solution of a variety of problems related to navigation channel maintenance.

A comparison of the environmental impacts of dredged material disposal with the geomorphic and hydraulic implications of open water disposal has revealed areas of serious conflict. In general, locations that constitute the most desirable disposal sites based on an analysis of the physical processes of the river system are undesirable when the biological processes are considered. Physically, the best locations for disposal are in regions where deposition would occur naturally. These include the downstream portion of point bars and other locations not directly subject to high velocities during either high stage or low stage flow. The man-induced depositional environment of the dike fields offers protected disposal sites, as does the interior of many alluvial islands. However, disposal in these locations usually involves serious and often unacceptable environmental impacts. The only remaining significant portion of the riverine environment that offers potential disposal locations is the main channel or thalweg region of the river itself.

A geomorphic analysis of the crossing and pool sequence supported by mathematical modeling of a particularly troublesome reach of Pool 25 on the Upper Mississippi indicates that dredging from a crossing and disposing the dredged material in a downstream pool can constitute a feasible alternative to the disposal problem. The process involves a degree of risk of impacting the integrity of the navigation channel downstream from the pool, particularly if dredging is followed by a small discharge hydrograph. However, the risks incurred would be outweighed by the potential environmental benefits at many locations. Based on data currently available, the direct biologic impacts of disposal in the main channel as well as possible secondary impacts from turbidity generated by the disposal process appear minimal. In addition, the serious ecological problems associated with open water disposal on marshlands and near chute channels, sloughs, and backwater areas are avoided by the process of thalweg disposal. Although conditions downstream of a proposed disposal site may preclude thalweg disposal at certain locations, in many cases disposing only a portion of the dredged material along the thalweg would still result in reduced environmental impacts. Consequently, the concept of thalweg disposal offers a viable alternative to both long term and emergency disposal requirements. The results of the limited analysis of this study are sufficiently promising to warrant additional investigation of the thalweg disposal concept.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The current widespread concern over the wise utilization of the nation's natural resources includes questions relative to the nature and significance of the environmental impact of dredging and disposal operations in the riverine environment. Despite growing concern over the environmental impact of dredging, it must be assumed that dredging of large volumes of material from the nation's rivers will continue, primarily because of a current lack of acceptable alternatives in creating and maintaining navigable waterways. The importance of water-borne commerce to the nation's economy supports this contention. Concurrently it must also be anticipated that increasing environmental concern will result in significant controls on dredging methods and disposal procedures. Solution to the dilemma of expanding requirements and growing constraints requires, in part, a fundamental understanding of the geomorphic and hydraulic response of a river system to the dredging process. In addition, the application of basic principles of river mechanics must be considered in developing more acceptable dredged material disposal techniques and in selecting disposal sites which minimize environmental impacts.

An analysis of the response of a river system to dredging and disposal operations requires establishing both the morphology and hydraulics of the system prior to man's influence, and the response to man's activities occurring prior to and concurrent with dredging. In the study area selected, a general description of the Pleistocene history of the region is essential to an understanding of the configuration and character of the natural river. Man's development of the Upper and Middle Mississippi River has not progressed in isolated, sequential steps. Construction of contraction works and locks and dams has taken place concurrently with dredging, and each has simultaneously affected, to some degree, the water and sediment transport characteristics of the system. Accordingly, an analysis of the combined effects of these development activities was a necessary prelude to an attempt to isolate the response of the system to a selected activity such as dredging and disposal operations. The specific aspects of the geomorphic scene which interrelate with the dredging process include: the crossing and pool sequence of both meandering and straight reaches, the island and side channel configuration of divided reaches, and the mechanics of bed-material transport.

The geomorphic analysis of the Upper and Middle Mississippi supports the following conclusions relative to change resulting from both natural processes and the combined effects of man's activities on the river:

1. The morphology of the modern Mississippi in the study area is primarily the result of Pleistocene history modified by tributary influence and by varying amounts of water and sediment delivered to the channel. The position of the Mississippi within its valley has remained essentially fixed during the last 150-200 years.

2. The use of contraction dikes to create a navigation channel produced a slight decrease in width between 1890 and 1930 on the Upper Mississippi, and a major decrease in width between 1890 and the present on the Middle Mississippi.

3. Subsequent to 1940, geomorphic analysis of the Upper Mississippi must be related to changes in a particular part of the lock and dam system. In the Pool 24, 25, and 26 detailed study area the immediate response to lock and dam construction was an increase in river surface width throughout a pool; however, the long term response has been a decrease in width immediately below a lock and dam and an increase in width just above a lock and dam.

4. The Mississippi above Cairo has experienced considerable within-channel change since 1890. Because the length of the Mississippi above Cairo has not changed appreciably, surface area change has generally mirrored the change in river width. Dike construction on the Middle Mississippi has produced significant decreases in island area and in riverbed area. On the Upper Mississippi the response has been more complex and a function of position in a navigation pool. Higher water levels immediately upstream of a lock and dam have produced decreased island area, while a lowering of bed elevations downstream from a lock and dam has resulted in lower stages and increased island areas.

5. Bed elevations on the Middle Mississippi have been lowered throughout the period of dike construction. The period of dike construction on the upper river was one of slight aggradation of the riverbed. The limited effectiveness of the low dikes constructed on the upper river, and the concentration of construction effort toward the end of the era of dike construction, coupled with a natural tendency toward aggradation on the Upper Mississippi contributed to this pattern of increasing bed elevations. This trend was reversed between 1930 and 1940 in the three pools of the detailed study area and general degradation has continued to the present in the lower two pools. Pool 24, however, has experienced general aggradation since 1940, indicating a tendency to trap incoming sediments from Pool 22 and the Salt River, and create a sediment deficient condition in the downstream pools. Local exceptions to these trends include some aggradation immediately above locks and dams and local scour below.

6. Viewing the river in cross section provides an integrated picture of the effects of changes in width, surface area, and bed elevation. In particular, the response to dike construction is clearly evident in the cross-sectional view. Flow area at St. Louis has been progressively decreased until it is now only two-thirds that of the natural river. A similar decrease has occurred all along the Middle Mississippi wherever the channel has been contracted. On the Upper Mississippi flow areas generally decreased during the period of dike construction and increased following lock and dam construction in response to the variation in surface width and bed elevation.

7. Geomorphic response of the Mississippi above Cario to man's activities is reflected in the hydraulic parameters of discharge and stage. Annual peak flood discharges on the upper river have remained on the average, unchanged through the period of record. On the middle river present day peak floods are, on the average, slightly lower than in the past, reflecting the construction of storage dams on the Missouri. Minimum flows have increased slightly both above and below the mouth of the Missouri. The effect on river stage has been more significant. At St. Louis, the decrease in both flow area and overbank storage has contributed to an increase in the annual maximum flood stage. Although present day floods on the Middle Mississippi produce flood stages higher than similar discharges produced in the past, levees prevent flood damage when the river exceeds bankfull

stage. On the Upper Mississippi minimum stages have been strongly influenced by man's development. Minimum stages have decreased at locations immediately below a lock and dam and increased sharply at locations above a lock and dam shortly after first full pool was reached at each location.

8. Sediment data supports the characterization of the Upper Mississippi as a clear water stream and the Mississippi below the Missouri as a heavy sediment carrier. Sediment records on the Mississippi above Cairo are not of sufficient length to permit an accurate determination of the effect of development on sediment load. Available data does suggest that sediment loads have been decreasing in the recent past. The sediment trapping effect of the upstream pools of the Upper Mississippi lock and dam system have certainly been a contributing factor to the observed general degradation in the lower pools of the system.

Several additional conclusions drawn from an analysis of historical changes of islands and side channels on the Middle Mississippi are pertinent to the dredging and disposal problem:

1. Natural backwater channels are a product of the natural, uncontrolled, shifting river. Any river subject to development will experience a deterioration of the natural backwater channels unless these channels are maintained artificially.

2. Unless steps are taken to prevent it, ultimately nearly all natural and man-made side channels will fill with sediment and become indistinguishable from the floodplain.

3. Small natural and man-made chute channels fill at a rate of up to three feet a year. Backwater channels fill at rates of between one and five inches a year. Those few large natural chute channels in existence today will remain open for a long period of time.

4. The life of a side channel can be increased if it can be isolated on the upper end from the main channel. When isolated, the side channel becomes a backwater channel and rates of sedimentation are small.

Because the construction of contraction works and navigation dams on the Mississippi has occurred concurrently with dredging to maintain the navigation channel, the task of isolating the response of the river to just the dredging process is difficult. Consequently, conclusions relative to this response are based, to some extent, on inference, deduction, and an analysis of secondary effects.

Contraction works and flow regulation by locks and dams are generally considered permanent solutions to the problem of providing navigable depths in a river system. In comparison, dredging must be classed as a temporary solution to the problem of maintaining a navigation channel. Dredging and other temporary solutions to the navigation problem have several distinct advantages over more permanent solutions. Temporary solutions are relatively simple and direct in their application. They also afford a degree of flexibility in meeting unexpected or changing requirements in a waterway system not possessed by more permanent solutions which require large capital investment and long-range planning. Temporary solutions, however, all attempt to change the local configuration of the channel without changing the forces that have produced that configuration, and for this reason can be expected to prevail against the dominant forces of the system for only a limited period of time. Field experience has shown that, although temporary, dredging is usually a necessary adjunct to programs for river system development by more permanent means.

While a variety of dredging devices have been proposed and tested on the Mississippi, it was only after the development of the hydraulic pipeline dredge in the 1890's that dredging assumed an important role in navigation channel maintenance. The cutterhead dredge and the dustpan dredge currently perform the bulk of the channel maintenance dredging in the study area. Each has a different mode of operations and different capabilities, however, each is well suited to navigation channel maintenance. Both the cutterhead and dustpan dredge discharge dredged material as a water-sediment slurry through a discharge pipeline; but, because of its slower rate of advance, the cutterhead dredge is better adapted to discharging overbank through shore pipe. In addition, the cutterhead dredge is better adapted to development dredging operations.

The dredging of a channel through a crossing or shoal can be considered successful if the cut achieves the required navigation depth, and if the cut does not require re-dredging during the navigation season. The depth, width, and alignment of the cut all influence its stability, and, thus, the degree of success achieved. Some indication of the effect of depth and width on the stability of a dredged cut can be obtained from a relatively simple hydraulic analysis. The influence of alignment on stability is best approached through field experience or a hydraulic model study.

The influence of changing the depth of a dredged cut while maintaining a constant width can be estimated from a simple hydraulic analysis. Under these conditions, bed shear, and thus contact-load transport capacity, varies inversely as the square of the depth, for a channel with constant width, roughness, and discharge. Thus, the policy of advanced maintenance dredging or overdepth dredging at constant width as currently practiced on many rivers in the United States can significantly decrease the transport capacity of the dredged cut and thus its stability. A policy of overdepth dredging can be supported only if the economic benefits that result from the availability of greater depths for longer periods, and from more efficient scheduling of dredging operations balance the added cost and the impact of disposing greater quantities of dredged material.

While a constant width analysis sheds some light on the stability problems associated with overdepth dredging, in the general case the influence of width and depth changing concurrently must be considered. Because velocity and, thus, transport capacity can be related to the hydraulic radius of the channel or dredged cut under conditions of steady, uniform flow, the impact of dredging on the hydraulic radius of a section can be used as a criteria for stability. Under these idealized flow conditions, design curves can be prepared to permit choosing values of width and depth which will produce a tendency toward stability in a dredged cut.

Although a steady, uniform flow analysis indicates that under optimum conditions a dredged cut can be designed to be stable, the influence of both alignment and unsteady flow on stability must also be considered. The results of test dredging programs reveal that dredged cuts can be extremely unstable under conditions of nonuniform, unsteady flow in a natural river. This instability results, in part, from the difficulty of aligning the dredged cut with both the high-stage and low-stage flow patterns of an alluvial river. The consequence of mal-alignment under either flow condition is obliteration of the dredged cut.

The impact of unsteady flow on the stability of a dredged cut can be evaluated by an examination of the sensitivity of a dredged cut to changing stage. Comparing river stage, average depth over crossings, and dredging requirements shows that, in general, crossings are built up and dredged cuts filled when river stage falls after a period of high flow. During a period of falling stage the available depth on crossings decreases rapidly unless dredging is initiated to counteract the trend. During an extended period of low flow, dredging requirements taper off, but increase again if the stage rises or fluctuates.

Although it may be possible to design a stable dredged channel for uniform, steady flow conditions, the changing conditions of flow on a natural river lead to the conclusion that a dredged cut must be considered successful if it survives a single navigation season.

Because of the relative instability of a dredged channel in an alluvial river, it must be concluded that the dredged cut itself does not significantly alter river morphology. However, the combined use of dredging, dikes, and disposal of dredged material in the dike fields can induce major changes in the cross-sectional characteristics of a river. The lateral redistribution of sediments from the main channel to dike fields on the channel periphery has produced a marked decrease in the width to depth ratio in many sections of the study area.

Channel shape in the form of the width to depth ratio has been related by many researchers to the capacity of a channel for the transport of both suspended and contact load. To the extent that dredging and contraction reduce the width to depth ratio at a section, they increase the channel's capacity for transport of the suspended portion of the bed-material load but decrease the capacity for transport of contact load. The lateral redistribution of sediment by dredging, when combined with contraction works, not only directly interrupts the natural downstream movement of contact-load sediments, but also affects local channel morphology and indirectly retards the movement of contact load through the system. Thus, dredging accentuates the sediment trapping effects of the navigation pools on the Upper Mississippi, and so is a contributor to the sediment deficient conditions noted in the lower parts of the navigation system.

Although dredging operations in the riverine environment are generally maintenance oriented, the dredge also provides the river engineer with a means of rapidly altering channel configuration and accelerating morphologic processes in support of river development programs. In this regard, dredging can be considered a morphologic agent responsive to engineering requirements. In the same context, the application of hydraulic and mechanical dredges to obtain gravel and other materials from the river for construction and related uses can seriously impact river morphology and stability by removing significant quantities of the coarser sediments from the river. This coarser fraction, particularly gravel, has a tendency, through hydraulic sorting, to armor the riverbed, thereby retarding or arresting excessive scour, stabilizing banks and bars, and preventing excessive sediment movement. Gravel armored sandbars serve as semipermanent controls that define river form. Removal of the gravel armor from such features can lead to erosion and loss of this control. Although the adverse impacts of gravel mining have been observed primarily on the Lower Mississippi, projected increases in the use of sand and gravel resources in the Upper Mississippi basin do suggest that gravel mining will eventually impact the morphology of the upper river.

Among the potential applications of the dredge as an agent for morphologic change are the dredging of openings into backwater areas and the enhancement of the recreational potential of selected alluvial islands. Dredging openings into backwater areas that have experienced a loss of circulation and reduction in water quality as a result of natural or man-induced deposition in feeder channels, can counter the processes of eutrophication. For favorably located side channels the procedure would have long range beneficial effects; but, the possibility for adverse impacts also exists. Converting a closed slough to an open chute channel could accelerate sedimentation rates, particularly if the opening is oriented to induce the flow of sediment-laden high stage flows through the opened channel. The use of selective disposal of dredged material to create open spaces on alluvial islands could greatly enhance their recreational potential. However, the habitat value of the stands of hardwoods which occupy the interior of many alluvial islands together with the susceptibility of these species to stem burial and the already severe impacts of agricultural activities on the river bottom forest, dictate against this procedure except where unusual recreational benefits will result.

A comparison of geomorphic data with records of dredging quantities required to obtain and maintain the 9-foot channel on the Mississippi provides a basis for determining those factors that influence dredging requirements on an alluvial river. Such non-engineering factors as project funding and dredge plant availability have influenced the records of dredged volumes, and limit, to a degree, the establishing of a direct cause and effect relation between dredged quantities and hydraulic or geomorphic change. However, increasing dredged volumes in the detailed study area on the Upper Mississippi appear closely related to:

1. Initial dredging requirements to make the transition from the 6-foot channel to the 9-foot channel.
2. Extended periods of abnormally low flow where lack of water in the system becomes a controlling factor.
3. Extended periods of unusually high flow.

4. Operational policies such as the practice of overdepth and overwidth dredging.

5. Effectiveness and efficiency of dredging operations.

The records of dredged volume on the Upper and Middle Mississippi also indicate that a considerable portion of the contact load moving though the system is being impacted by the dredging process. Annual dredged volume in the three pools of the detailed study area on the Upper Mississippi represent about 57 percent of the contact load moving through this reach annually. On the Middle Mississippi approximately 34 percent of the annual contact load is impacted by the dredging process.

On the Middle Mississippi where relationships under conditions of open river regulation are much less complex, a tendency toward decreased dredged volumes can be directly related to contraction efforts. Dredging requirements in a short study reach of the Middle Mississippi have been completely eliminated by carrying contraction efforts to an extreme. Contraction efforts on the Middle Mississippi have, however, been partially responsible for a significant increase in river stage for the higher discharges. It must be concluded that increased contraction to minimize dredging can be applied only so long as secondary effects in terms of hydraulic and geomorphic change, are acceptable. Beyond this point, continued maintenance dredging must be accepted.

Field experience has established that most dredging operations are conducted to improve depths on the shallow crossings and to increase channel width adjacent to point bars in bends. Plots of dredging frequency and dredged volume by location provide a means of relating dredging requirements to other geomorphic characteristics. The transition from open river regulation by contraction dikes to low-flow regulation by navigation dams on the Upper Mississippi radically altered the distribution of dredging locations. Dredging requirements which had been distributed rather evenly through the detailed study area were re-ordered following lock and dam construction with a concentration of higher dredging frequency and larger dredged volumes at a reduced number of dredging sites.

Dredging trouble spots, that is, areas of high dredging frequency or large dredged volume in the Pool 24, 25, and 26 detailed study area, can be closely related to the following factors:

1. Location of locks and dams.
2. Dredging location relative to pool primary control point.
3. Straight reaches.
4. Divided reaches.

Combinations of these factors produce particularly troublesome reaches. In the detailed study area the most serious dredging problems occur in straight reaches which are located above the pool primary control point, and which are also divided by alluvial islands. In the reach studied, dredging requirements did not correlate closely with tributary locations; however, this is a factor in other areas.

The application of geomorphic indicators of dredging problem areas to a typical field problem establishes their problem solving potential and value to the river engineer. The case study selected involved the identification of dredging sites for inclusion in a pilot program to assess the effects of reduced overdepth dredging on the navigation channel of the Upper Mississippi. Based on the favorable results of this case study, it can be concluded that an analysis of the hydraulic and geomorphic response of the river to dredging provides the river engineer with a methodology applicable to the solution of a variety of problems related to navigation channel maintenance.

A comparison of the environmental impacts of dredged material disposal with the geomorphic and hydraulic implications of open water disposal reveals areas of serious conflict. In general, locations that constitute the most desirable disposal sites, based on an analysis of the physical processes of the river system, are undesirable when the biological processes are considered. Physically, the best locations for disposal are in regions where deposition would occur naturally. These include the downstream portion of point bars and other locations not directly subject to high velocities during either high-stage or low-stage flow. The man-induced depositional environment of the dike fields offers protected disposal sites, as does the interior of many alluvial islands. However, disposal in these locations usually involves serious and often unacceptable environmental impacts.

From an environmental point of view, open water disposal of dredged material is usually undesirable under the following specific conditions:

1. When dredged material is disposed in such a manner as to cause filling of chutes and side channels.

2. When dredged material is disposed in or near feeder channels to sloughs and backwater areas.

3. When dredged material is disposed on submerged contraction dikes.

4. When dredged material is disposed at the upstream end of islands.

A hydraulic model study confirms the undesirability of disposing dredged material at the upstream end of an island. Geomorphic analysis of the development and evolution of islands and side channels on the Middle Mississippi suggests, however, that in some cases the closing of a chute channel can prolong the life of the chute by converting it to a slough where sedimentation rates are generally slower than in an open chute.

A number of alternatives to the current practice of open water disposal in the riverine environment exist and are being studied. Unfortunately, many of these alternatives involve major modification or additions to the existing dredging plant, or major changes in current dredging operations. In contrast, the disposal of dredged material in the main channel or thalweg region of the river offers an alternative which is based on the morphology and hydraulics of the river system, which avoids the direct biologic impacts of bankline or island disposal, and which is generally within the capabilities of existing dredge plant.

A geomorphic analysis of the crossing and pool sequence supported by mathematical modeling of a particularly troublesome reach of Pool 25 on the Upper Mississippi leads to the following conclusions:

1. Dredging from a crossing and disposing the dredged material in a downstream pool constitutes a feasible alternative to the disposal problem.

2. The process of thalweg disposal of dredged material involves a degree of risk of impacting the integrity of the navigation channel downstream from the pool, particularly if dredging is followed by a small discharge hydrograph. However, the risks incurred would be outweighed by the potential environmental benefits at many locations.

3. Based on data currently available, the direct biologic impacts of disposal in the main channel, as well as possible secondary impacts from turbidity generated by the disposal process, appear minimal. In addition, the serious ecological problems associated with open water disposal on marshlands and near chute channels, sloughs, and backwater areas are avoided by the process of thalweg disposal.

4. Conditions downstream of a proposed disposal site may preclude complete reliance on thalweg disposal at certain locations, but in many cases disposing only a portion of the dredged material along the thalweg would still result in reduced environmental impacts.

5. The concept of thalweg disposal offers a viable alternative to both long-term and emergency disposal requirements. As an emergency procedure where unexpected shoaling threatens the integrity of the navigation channel and immediate corrective dredging is required, thalweg disposal holds considerable promise. Long-term implementation of the practice should be preceded by a detailed analysis of the hydraulics and morphology of the specific reach involved.

6. The results of the limited analysis of this study are sufficiently promising to warrant additional investigation of the concept of thalweg disposal of dredged material

6.2 Recommendations

The data gathering, research, and analysis required for this study have indicated that the Corps of Engineer's riverine maintenance dredging program could benefit from modification and supplementation in several areas. Data gathering efforts reveal a definite lack of post-dredging data beyond that required to estimate quantities actually dredged. Surveys to evaluate the performance of a dredged cut throughout a navigation season would be invaluable to an investigation of the stability of dredged cuts at various locations and alignments in a channel. With the notable exception of the St. Paul District's efforts to evaluate the effects of reduced overdepth dredging by documenting channel conditions at the end of the navigation season, such data was difficult to obtain. A long term post-dredging survey program and documentation effort in all Districts concerned with riverine maintenance dredging would provide the data base for a thorough examination of the effects of depth, width, alignment, and location of a dredged cut on channel stability under a wide variety of hydraulic and geomorphic conditions. Such an analysis could be instrumental in reducing both the frequency and volume of dredging at specific locations, which in turn would reduce environmental impacts.

The lack of test dredging data to support this study was striking. With the exception of Ockerson's description of test dredging on the Lower Point Pleasant Bar in 1898, and Somervell's summary of the 1930-31 test dredging program conducted by the Memphis District, the results of experimental dredging programs have evidently not been adequately documented or widely distributed. Because of the complexity of alluvial channel flow and its interaction with a dredged channel, prototype test dredging offers perhaps the most reliable method of evaluating proposed modifications to existing dredging and disposal procedures. Experimental programs such as the Portland District's testing of a cut and fill dredging process provide data on which to base decisions concerning modifications to existing practice. In the same light, programs such as the St. Paul District's evaluation of turbidity effects at Crosby Island, the investigation of the feasibility of opening a backwater area at Mule Bend, and the reduced overdepth dredging pilot program should be encouraged and funded.

While physical model studies can provide valuable information to supplement prototype experimentation, there are only a few model studies available in the literature in which dredging and disposal procedures were more than a peripheral aspect of the problem under investigation. The notable exception is the Waterways Experiment Station's 1941 study of shoaling problems in the Manchester Islands reach of the Ohio River. The extent to which this hydraulic model investigation contributed to the analysis of the geomorphic and hydraulic response of a river to dredging is indicative of the value of such studies. The application of physical modeling to investigate dredging and disposal problems at specific sites should be expanded.

The use of a mathematical model to investigate the feasibility of thalweg disposal of dredged material has demonstrated its applicability to the solution of dredging and disposal problems. It was concluded that, based on the limited analysis of this study, the concept of thalweg disposal offers a viable alternative to both long term and emergency disposal requirements. Further application of mathematical modeling under a wider variety of geomorphic and hydrologic conditions is recommended to refine the conclusions of this study. Physical modeling of the thalweg disposal concept for specific reaches would provide a valuable supplement to mathematical modeling and would offset, to a degree, the one-dimensional limitations of the mathematical model used in this study.

Solution to the conflicting demands of expanded use of the nation's navigable waterways, and consequent continued dredging requirements, with growing environmental concerns and constraints requires the development of new dredging and disposal techniques. Improved documentation of maintenance dredging activities, expanded prototype experimental dredging programs, and increased use of physical and mathematical modeling procedures are strongly recommended as a means of providing the necessary baseline information for the development of such techniques.

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APPENDIX A

TABLES

TABLE 1

Annual Disposal Requirements Related to Corps of Engineers Maintenance Programs (millions of cubic yards)

	Environmental Region	Water Type	Disposal Category
Open water disposal total			200
Fresh water		110	
Lacustrine (Lake)	6		
Riverine	104		
Salt water		90	
Estuarine and Marine	90		
Land disposal (confined and unconfined)			100
Total			300

TABLE 2 Geometry of Santa Fe Chute*

	1884	1968
Surface Area	0.68 sq mi	0.73 sq mi
Length	2.85 mi	3.90 mi
Average Width	1270 ft	990 ft

*(after Simons et al., 1974)

TABLE 3 Classification for Determining Bed Load*

Concentration of Suspended Load (ppm)	Type of Material Forming the Channel of the Stream	Texture of Suspended Material	Percent Bed Load in Terms of Measured Suspended Load (pct)
Less than 1000	Sand	Similar to bed material	25 to 150
Less than 1000	Gravel, rock, or consolidated clay	Small amount of sand	5 to 12
1000 to 7500	Sand	Similar to bed material	10 to 35
1000 to 7500	Gravel, rock, or consolidated clay	25 percent sand or less	5 to 12
Over 7500	Sand	Similar to bed material	5 to 15
Over 7500	Gravel, rock, or consolidated clay	25 percent sand or less	2 to 8

*(after Lane and Borland, 1951)

TABLE 4 Classification of Alluvial Channels*

Mode of Sediment Transport and Type of Channel	Channel Sediment (M) Percent	Bedload (Percentage of Total Load)	Channel Stability: Stable (Graded Stream)	Channel Stability: Depositing (Excess Load)	Channel Stability: Eroding (Deficiency of Load)
Suspended Load	>20	<3	Stable suspended-load channel. Width-depth ratio less than 10; sinuosity usually greater than 2.0; gradient relatively gentle.	Depositing suspended load channel. Major deposition on banks cause narrowing of channel; initial streambed deposition minor.	Eroding suspended-load channel. Streambed erosion predominant; initial channel widening minor.
Mixed Load	5-20	3-11	Stable mixed-load channel. Width-depth ratio greater than 10 less than 40; sinuosity usually less than 2.0 greater than 1.3; gradient moderate.	Depositing mixed-load channel. Initial major deposition on banks followed by streambed deposition.	Eroding mixed-load channel. Initial streambed erosion followed by channel widening.
Bedload	<5	>11	Stable bedload channel. Width-depth ratio greater than 40; sinuosity, usually less than 1.3; gradient relatively steep.	Depositing bedload channel. Streambed deposition and island formation.	Eroding bedload channel. Little streambed erosion; channel widening predominant.

*(after Schumm, 1971)

TABLE 5 Average River Surface Widths*

Pool 24

Location	Average Width, ft 1891	1927	1940	1973
Upper 1/4	4500	4200	4300	4100
Middle 1/2	4500	4000	4300	3800
Lower 1/4	4100	4000	4300	4200
Average for Pool 24	4400	4100	4300	4000

*The 1895 and 1940 widths were scaled from topographic maps prepared by The Corps of Engineers.

The 1927 and 1973 widths were scaled from uncontrolled aerial Photo mosaics.

TABLE 6 Change in Riverbed Elevations (in Feet)

	1891 to 1930		1891 to 1940		1940 to 1971	
	Average Bed Elevation	Thalweg Bed Elevation	Average Bed Elevation	Thalweg Bed Elevation	Average Bed Elevation	Thalweg Bed Elevation
Pool 24						
Upper 1/4	+1.5	+0.2	-0.8	-6.0	+1.9	+1.7
Middle 1/2	-0.2	-0.6	-2.5	-5.6	+2.0	+2.7
Lower 1/4	+1.3	+0.8	+1.2	-4.1	+2.7	+4.4
Pool 25						
Upper 1/4	+1.2	+2.4	-4.3	-1.8	-2.5	-1.7
Middle 1/2	+0.1	-1.7	-3.5	-5.1	+0.9	+0.8
Lower 1/4	+0.4	+0.6	-0.8	-0.8	-2.7	-3.3

+ bed elevation up

- bed elevation down

TABLE 7 Cross Section Data

Location	Date Year	Stage ft above msl	Area sq ft	Top Width ft	Area / Top Width (Av. Depth) ft	Mean Bottom Elevation ft above msl
			Pool 24			
Cottel Island	1891	446	17080	2000	8.54	437.46
(RM 300)	1930	446	12800	3120	4.10	441.90
	1971	446	16880	1710	9.87	436.13
			Pool 25			
Eagle Island	1891	432	14000	2615	5.35	426.65
(RM 270)	1930	432	12350	2380	5.19	426.81
	1971	432	17500	2290	7.64	424.36
			Pool 26			
Turkey Island	1891	414	15800	2670	5.92	408.08
(RM 236.9)	1930	414	11360	2030	5.60	408.40
	1971	414	18600	1970	9.44	404.56

TABLE 8 Trends in Annual Discharges and Stages

Location	Maximum Stage	Minimum Stage	Maximum Discharge	Average Discharge	Minimum Discharge
Mississippi River at Alton	None	Down	None	Up	Up
Mississippi River at Keokuk	None	Down	None	Down	Up
Mississippi River at Hannibal	Up	Up			
Mississippi River at Grafton	None	Up			

TABLE 9 Average River Surface Widths

(a) History of River Widths - Middle Mississippi*

Year	Average Width ft
1821	3600
1888	5300
1968	3200

(b) River Widths at St. Louis*

Year	Width ft
1803	3100
1808	3200
1837	3700
1843	3900
1849	4200
1888	2100
1973	2100

*(after Simons et al., 1974)

TABLE 10 Surface Area Change--Middle Mississippi*

Year	Surface Area sq mi	Island Area sq mi	Riverbed Area sq mi
1821	109	14	95
1888	163	35	128
1968	100	17	83

*(after Simons et al., 1974)

TABLE 11 The Top-Ten Flood Discharges at St. Louis*

Rank	Peak Discharge cfs	Year	Stage Rank
1	1,300,000	1844	2
2	1,054,000	1858	8
3	1,050,000	1855	9
4	1,040,000	1903	7
5	1,022,000	1851	10
6	926,000	1892	-
7	889,000	1927	-
8	863,000	1883	-
9	861,000	1909	-
10	855,000	1973	1

Rank	Maximum Stage ft	Year	Discharge Rank
1	43.3	1973	10
2	41.3	1844	1
3	40.2	1947	-
4	40.2	1951	-
5	39.0	1944	-
6	38.9	1943	-
7	38.0	1903	4
8	37.2	1858	2
9	37.1	1855	3
10	36.6	1851	5

*The period of record is 1843 to 1973
(after Simons et al., 1974)

TABLE 12 Yearly Suspended-Sediment Discharge of Missouri and Mississippi Rivers, 1949-63 (thousands of tons)*

Water Year	Mo. R. at Hermann	Miss. R. at Hannibal	Hermann plus Hannibal	Miss. R. at St. Louis
1949	328,400	8,700	337,100	282,300
1950	297,200	20,800	318,000	330,000
1951	423,400	59,800	483,200	417,200
1952	255,900	39,800	295,700	250,200
1953	94,600	15,900	110,500	99,600
1954	68,900	12,400	81,300	70,600
1955	65,800	9,400	75,200	74,500
1956	42,000	4,600	46,600	37,400
1957	66,800	5,000	71,800	74,800
1958	149,300	3,300	152,600	108,100
1959	99,100	10,200	109,300	111,200
1960	122,100	53,100	175,200	187,700
1961	124,200	13,700	137,900	142,800
1962	135,800	42,600	178,400	152,200
1963	65,500	7,000	72,500	74,300
Total	2,339,000	306,300	2,645,300	2,412,900
Average	155,900	20,400	176,400	160,900

*(after Jordan, 1968)

TABLE 13 Values of Hydraulic Radius for Combinations of n and Δ *

Δ/n	1	2	3	6	10	20	50	100	∞
1	5.67	8.50	10.20	12.75	14.17	15.45	16.35	16.67	17
2	6.00	9.00	10.80	13.50	15.00	16.36	17.31	17.65	18
3	6.33	9.50	11.40	14.25	15.83	17.27	18.27	18.63	19
5	7.00	10.50	12.60	15.75	17.50	19.09	20.19	20.59	21
7	7.67	11.50	13.80	17.25	19.17	20.91	22.12	22.55	23
9	8.33	12.50	15.00	18.75	20.83	22.73	24.04	24.51	25
12	9.33	14.00	16.80	21.00	23.33	25.45	26.92	27.45	28

Line of nΔ = 32

*Undisturbed Depth, (D), = 16 feet

TABLE 14 Discharge Distribution--Manchester Islands Model Study*

Plan	Stage	Total Discharge Ohio River c.f.s.	Discharge Distribution Past Manchaster Islands: Kentucky Channel Discharge c.f.s.	Kentucky Channel Percent of total	Middle Channel Discharge c.f.s.	Middle Channel Percent of total	Ohio Channel Discharge c.f.s.	Ohio Channel Percent of total
Base Test	4	99,400	51,700	52	19,900	20	27,800	28
	8	130,700	66,600	51	28,800	22	35,300	27
	15	190,000	93,000	49	47,500	25	49,500	26
A	4	99,400	69,600	70	*	*	29,800	30
	8	130,700	73,200	56	13,100	10	44,400	34
	15	190,000	95,000	50	41,800	22	53,200	28
B	4	99,400	*	*	61,600	62	37,800	38
	8	130,700	27,500	21	61,400	47	41,800	32
	15	190,000	70,500	37	70,500	37	49,000	26
C	4	99,400	18,500	19	30,800	31	50,100	50
	8	130,700	47,000	36	30,000	23	53,700	41
	15	190,000	76,000	40	52,000	27	62,000	33

*No flow in channel during this stage.

*(after Corps of Engineers, Waterways Experiment Station, 1941)

TABLE 15 Velocity Observations--Manchester Islands Model Study

Plan	Stage	Velocity in ft. per sec.*					
		Kentucky Channel		Middle Channel		Ohio Channel	
		Ave.	Max.	Ave.	Max.	Ave.	Max.
Base Test	4	3.5	5.5	3.7	5.6	3.7	5.4
	8	3.6	5.0	3.8	5.7	3.6	4.9
	15	3.7	5.4	3.8	5.7	3.6	5.0
A	4	4.5	6.3	**	**	3.4	5.4
	8	3.9	5.1	1.6	2.4	3.5	4.9
	15	3.7	5.0	3.0	4.6	3.6	5.0
B	4	**	**	6.4	8.9	4.6	6.4
	8	1.5	3.4	4.9	6.3	4.3	6.3
	15	2.7	4.2	4.2	5.5	3.4	5.0
C	4	1.2	2.7	5.7	9.9	4.9	7.3
	8	2.5	3.9	3.8	6.4	4.4	6.1
	15	3.0	4.8	4.3	6.2	3.7	7.3

*Velocity converted to prototype dimensions by means of model-velocity scale.

**No flow in channel during this stage.
(after Corps of Engineers, Waterways Experiment Station, 1941)

TABLE 16 Relation of Stage to Depth on Crossings and Dredging Requirements*

10-Day Periods		Average Stage at Memphis	Rising or Falling (at End of Period)	Average Depth of Bar at Crossings (below M.L.W.)	Channel Dredges Mobil-ized	Cubic Yards Moved
Hickman, Ky. to Memphis, Tenn.						
June	11-20	9.3	Rising	8.0	2	264,904
	21-30	12.6	Falling	7.0	2	7,360
July	1-10	10.0	Falling	7.7	2	185,970
	11-20	6.9	Falling	8.1	3	134,837
	21-31	4.4	Falling	9.2	5	595,045
Aug.	1-10	2.7	Falling	10.4	6	806,641
	11-20	2.0	Stationary	10.9	5	612,369
	21-31	2.1	Stationary	11.7	5	483,797
Sept.	1-10	1.9	Rising	12.0	6	421,036
	11-20	3.0	Rising	12.4	6	147,933
	21-30	3.4	Falling	12.1	5	13,888
Oct.	1-10	1.7	Falling	12.6	5	354,714
	11-20	1.4	Rising	12.5	5	258,466
	21-31	1.6	Falling	12.5	5	332,651
Nov.	1-10	1.3	Stationary	13.2	5	153,522
	11-20	1.2	Rising	12.7	5	381,218
	21-30	2.2	Rising	12.8	5	134,488
Dec.	1-10	2.9	Rising	11.3	5	234,010
	11-20	3.9	Rising	10.4	3	18,103

Total - 5,540,952

*(after Somervell, 1932)

TABLE 17 Cross-Sectional Data at Locations with Dikes and Dredging

Location	Date Year	Stage above msl	Area sq feet	Top Width feet	Av. Depth $\left(\frac{\text{Area}}{\text{Top Width}}\right)$	$\frac{\text{Width}}{\text{Depth}}$
RM 229.2	1891	412	20,200	3229	6.26	516
Two Branch Island	1940	412	15,200	1660	9.16	181
RM 237.9	1891	414	18,400	2547	7.22	353
Turkey Island	1940	414	6,160	985	6.25	158
RM 270	1891	432	14,000	2615	5.35	473
Eagle Island	1940	432	11,800	1490	7.92	188
RM 273	1891	434	13,200	1771	7.45	238
Clarksville Island	1940	434	14,120	1380	10.23	135
RM 280	1891	437	18,250	3235	5.64	574
Crider Island	1940	437	28,400	2825	10.05	281
RM 290	1891	442	15,200	2292	6.63	346
Cottonwood Island	1940	442	13,360	1475	9.06	163
RM 300	1891	446	16,400	1710	9.59	178
Cottel Island	1940	446	11,200	1070	10.47	102

Average Percent Decrease in $\frac{W}{D} = 53\%$

TABLE 18 Cross-Sectional Data--Middleton and Angle Islands

Location	Date year	Stage above msl	Area sq feet	Top Width feet	Av. Depth $\left(\frac{\text{Area}}{\text{Top Width}}\right)$	$\frac{\text{Width}}{\text{Depth}}$
RM 275 Middleton Island	1891	434	18,000	2000	9.00	222
	1940	434	18,400	2590	7.10	365
RM 285 Angle Island	1891	439	15,680	2458	6.38	385
	1940	439	16,920	2500	6.77	369

TABLE 19 Average Annual Dredging by Pool

Before and After Lock and Dam Construction (cubic yards)

Pool	1906 - 1938	1949 - 1973	Percent Change
24	114,000	89,000	-22%
25	239,000	396,000	+66%
26	382,000	382,000	-

TABLE 20 Dredging Requirements--Middle Mississippi Prototype Reach (1000's cubic yards)

Location (RM)	63	64	65	66	67	68	69-74	Total
	Year							
154								
153			203.6	346.2	63.3			613.1
152	37.3				69.2			106.5
151				34.1	146.0			180.1
150								
149								
148	53.9	32.9	236.3	170.1				493.2
147	42.3				86.8			129.1
146								
145	154.7	168.7	73.4					396.8
144		132.4		141.6				274.0
143				181.2				181.2
142		39.3						39.3
141	95.0	169.1	85.0	58.7	62.5	264.0		734.3
140								
Total	383.2	542.4	598.3	931.9	427.8	264.0	0	3147.6

Start Construction Prototype Reach

End Construction Prototype Reach

TABLE 21 Selection Criteria for Reduced Overdepth Dredging Pilot Program

Location			Type of Reach			High Flow Alignment WRT* Low Flow Alignment			Location of Cut WRT* Thalweg			Projected Depth	Projected Volume	Site Recommended for Pilot Program		Average Depth in Cut and Change			Dredging Frequency
Site	River Mile	WRT* L&D or Tributary	Bend	Straight	Divided	Good	Average	Poor	Good	Average	Poor	(ft)	(cy)	Yes	No	Before Dredge	Just After	3-4 mo After	(1956-1974)
Coulters Island	802	5 mi above L&D #3		✓		✓			on thalweg			11	29,000	✓		12.34	14.19 +1.85	-	4
Reads Landing	763	.3 mi below Chippewa R.		✓				✓	on thalweg			12	60,500		✓	14.74	16.07 +1.33	17.39 +2.65	12
Crats Island	759	4.3 mi below Chippewa R.		✓	✓			✓	on thalweg			12	29,000		✓	11.59	12.52 + .93	13.20 +1.61	14
Teepeeota Pt. (cut 1)	757.7	5 mi above L&D #4			✓			✓			on point bar	11	17,000		✓	11.04	12.24 +1.20	13.24 +2.20	13
Beef Slough	754	2 mi above L&D #4	✓		✓		✓				on point bar	11	12,000		✓	10.48	12.18 +1.70	15.78 +3.30	7
Fisher Island	754.4	2 mi above L&D #5		✓	✓			✓			on point bar	11	7,300		✓	12.15	12.91 + .76	-	12
Wilds Bend	730.4	2 mi above L&D #5A	✓				✓			on point bar		11	17,000	✓		10.40	12.34 +1.94	-	6
Winters Ldg (cut 1 & 2)	708.8	6 mi below L&D #6	✓				✓		on thalweg			12	21,500	✓		11.75	13.21 +1.46	14.41 +2.66	3
Winters Ldg (cut 3)	708.5	6 mi below L&D #6	✓				✓				on point bar	12	59,000	✓		11.15	12.91 +1.76	12.67 +1.15	5
Brownsville	690.4	6 mi above L&D #8			✓			✓	on thalweg			11	11,000		✓	11.56	11.84 + .28	-	12

*WRT = with respect to

TABLE 22 Fate of Dredged Material in a Typical Divided Reach*

Disposal Location	Stable		Fate of Dredged Material	Location Recommended for Disposal	
	High Stage	Low Stage		yes	no
Along Ohio Bank above Mile 395	No	Yes	Moves through Ohio channel and forms bar below Island No. 2		✓
Along Kentucky bank above Mile 395	No	Yes	Shoals navigation channel at head and foot of Island No. 1		✓
Along Kentucky bank between Mile 394.8 and 395.3	Yes	Yes	Some low stage scour	✓	
Middle channel	No	No	Rapidly scoured and deposits in bars below Islands No. 1 and 2.		
Along south bank of Island No. 2 below mile 396	Yes	Yes	Most stable disposal site in reach	✓	
Head of Island No. 1	No	No	Low and high stages move material into both Kentucky and middle channels. Deposits in bars below Island No. 1 and along Island No. 2 below mile 396.		✓
South bank of Island No. 1	No	No	Material slowly erodes and deposits on bar at foot of Island No. 1		✓
Along Kentucky bank between mile 395.8 and 396	Yes	No	Moves along Kentucky bank and deposits on next crossing downstream		✓
Along Kentucky bank below mile 396	No	Yes	Moves along Kentucky bank and deposits on next crossing downstream.		✓

*Refer to Figure 83

APPENDIX B

FIGURES

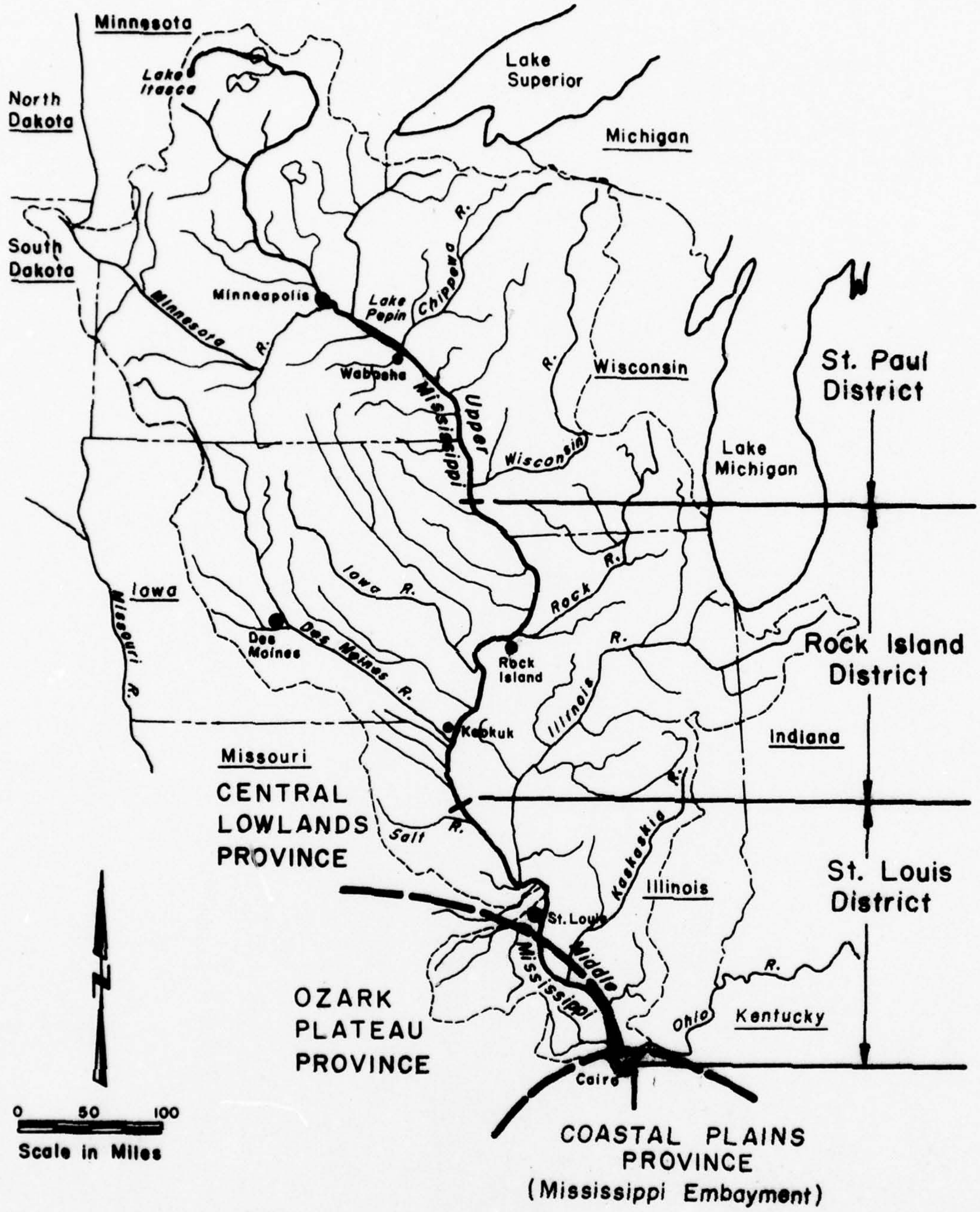


FIGURE 1. Upper and Middle Mississippi River Basin--Provinces, Tributaries, and Corps of Engineers Responsibility by District.

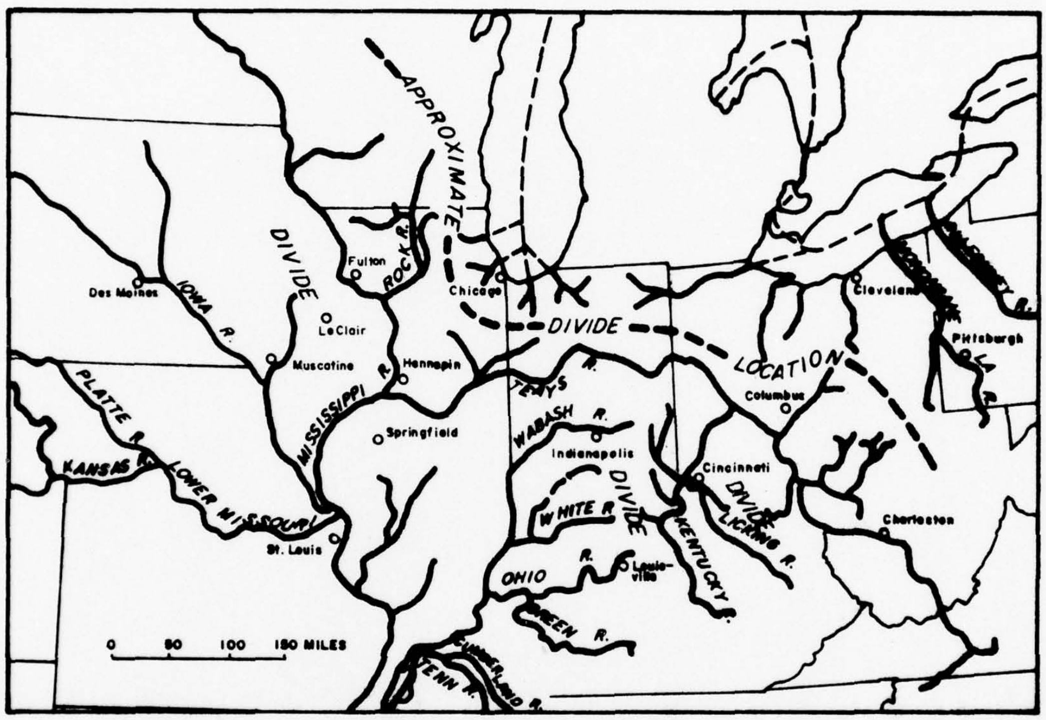


FIGURE 2. Preglacial Map of the River Systems
(after Simons et al., 1975).

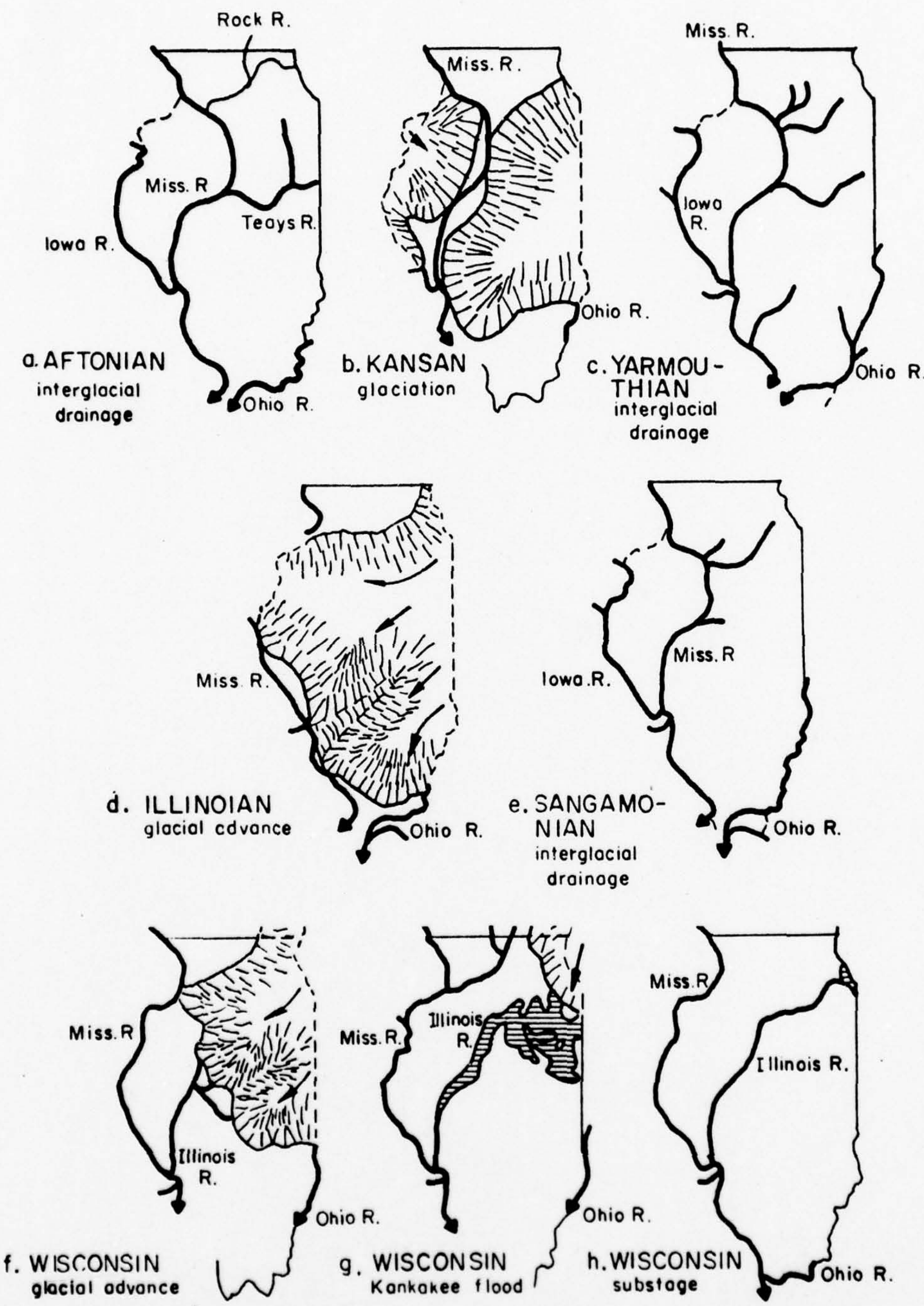


FIGURE 3. Pleistocene Changes--Mississippi River (after Frye et al., 1965).

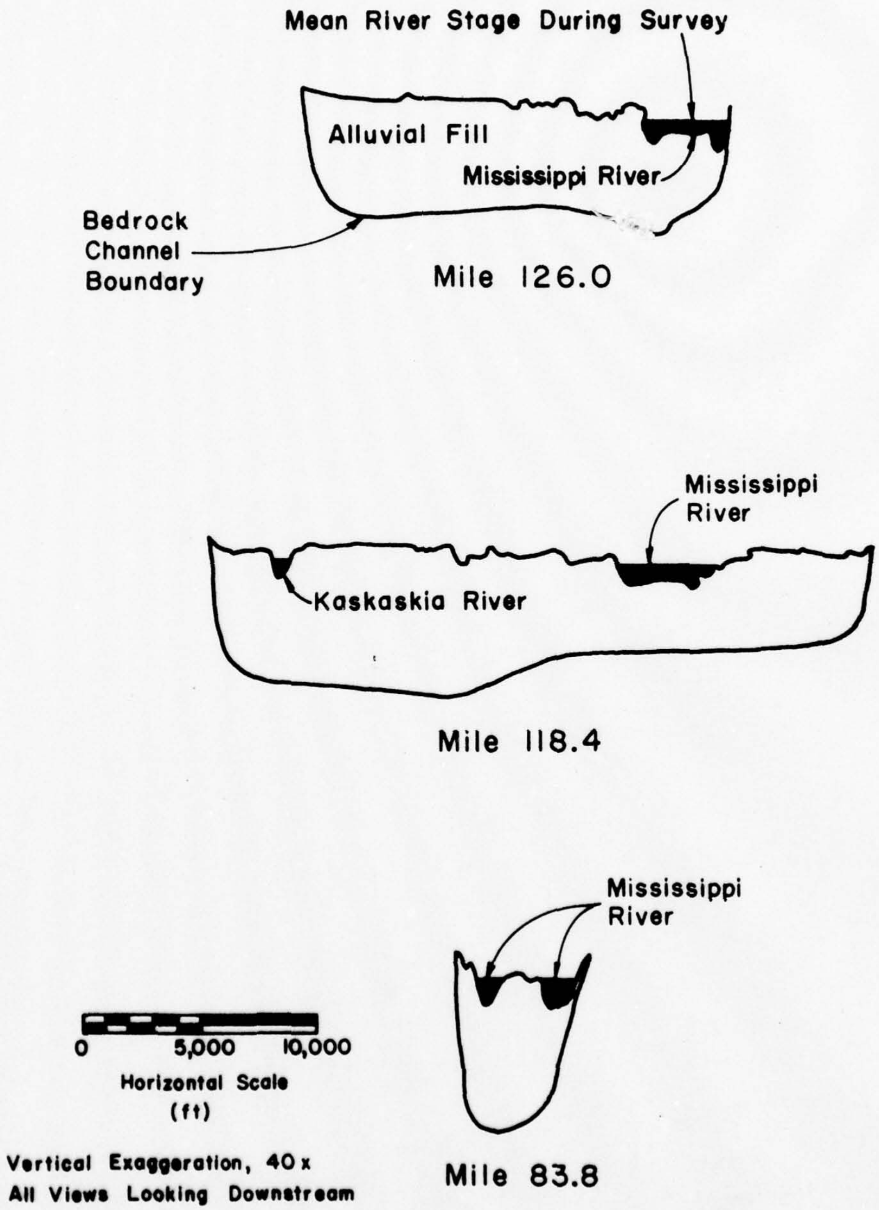


FIGURE 4. Middle Mississippi Valley Bedrock Cross Sections (after Degenhardt, 1973).

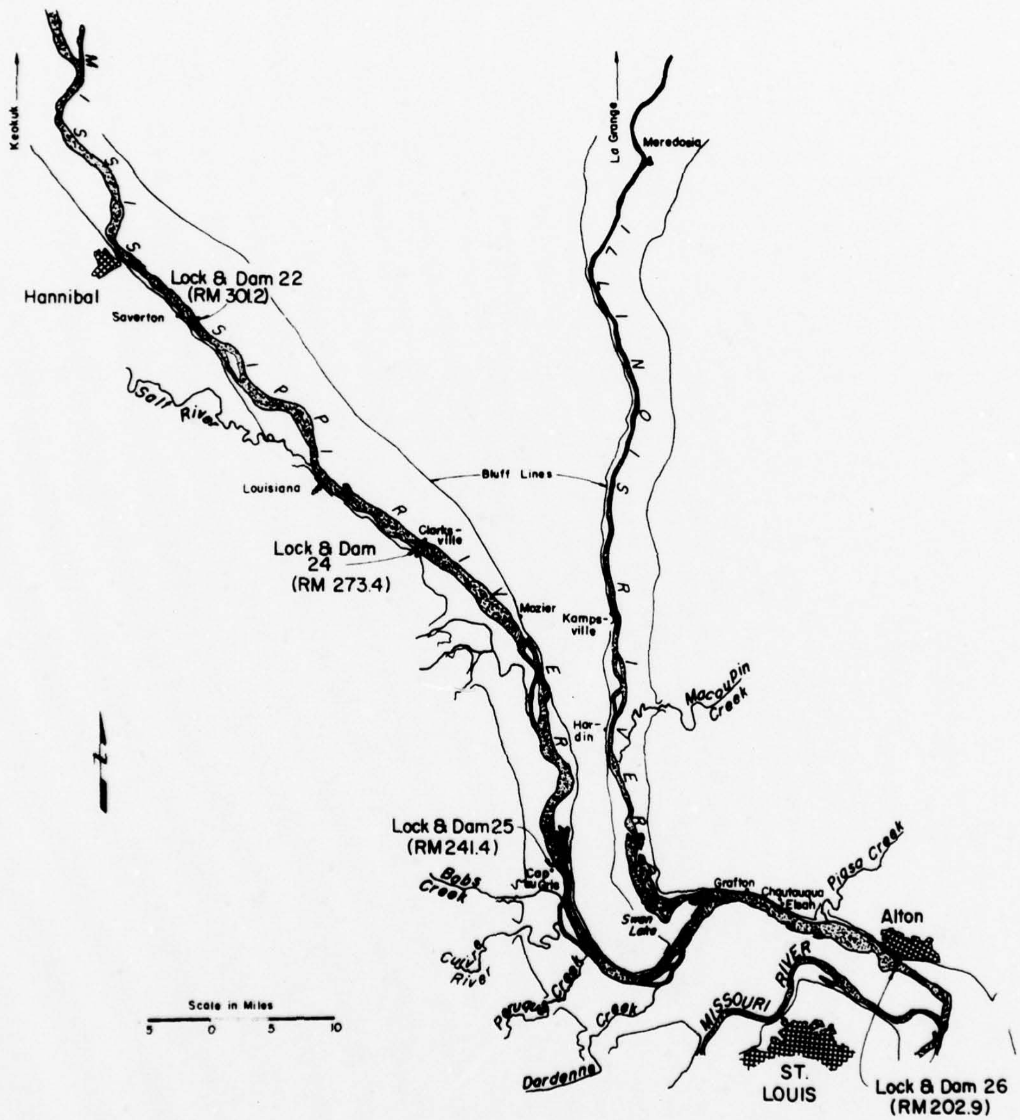


FIGURE 5. The Upper Mississippi River in the St. Louis District.

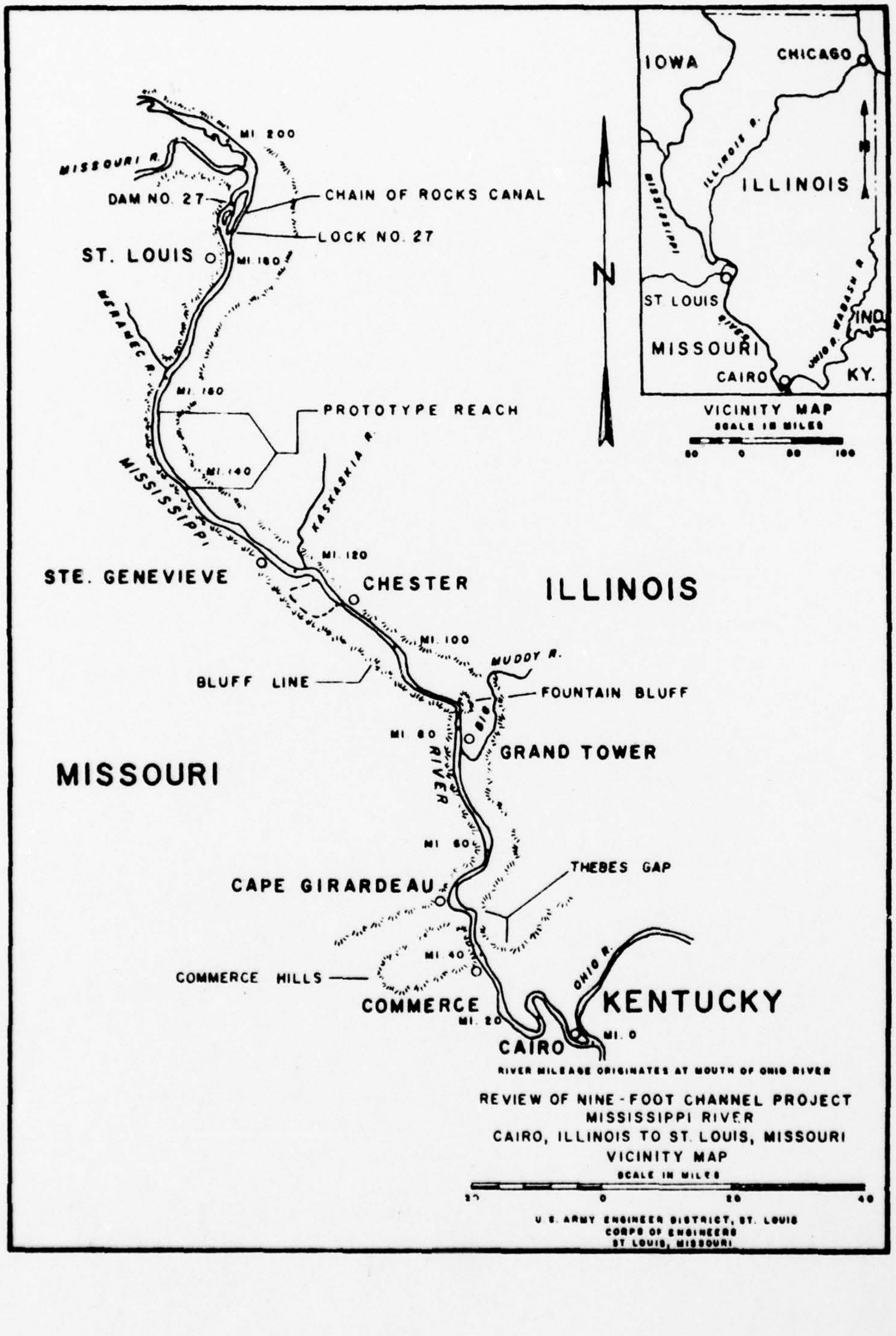


FIGURE 6. The Middle Mississippi River in the St. Louis District (after Degenhardt, 1973).

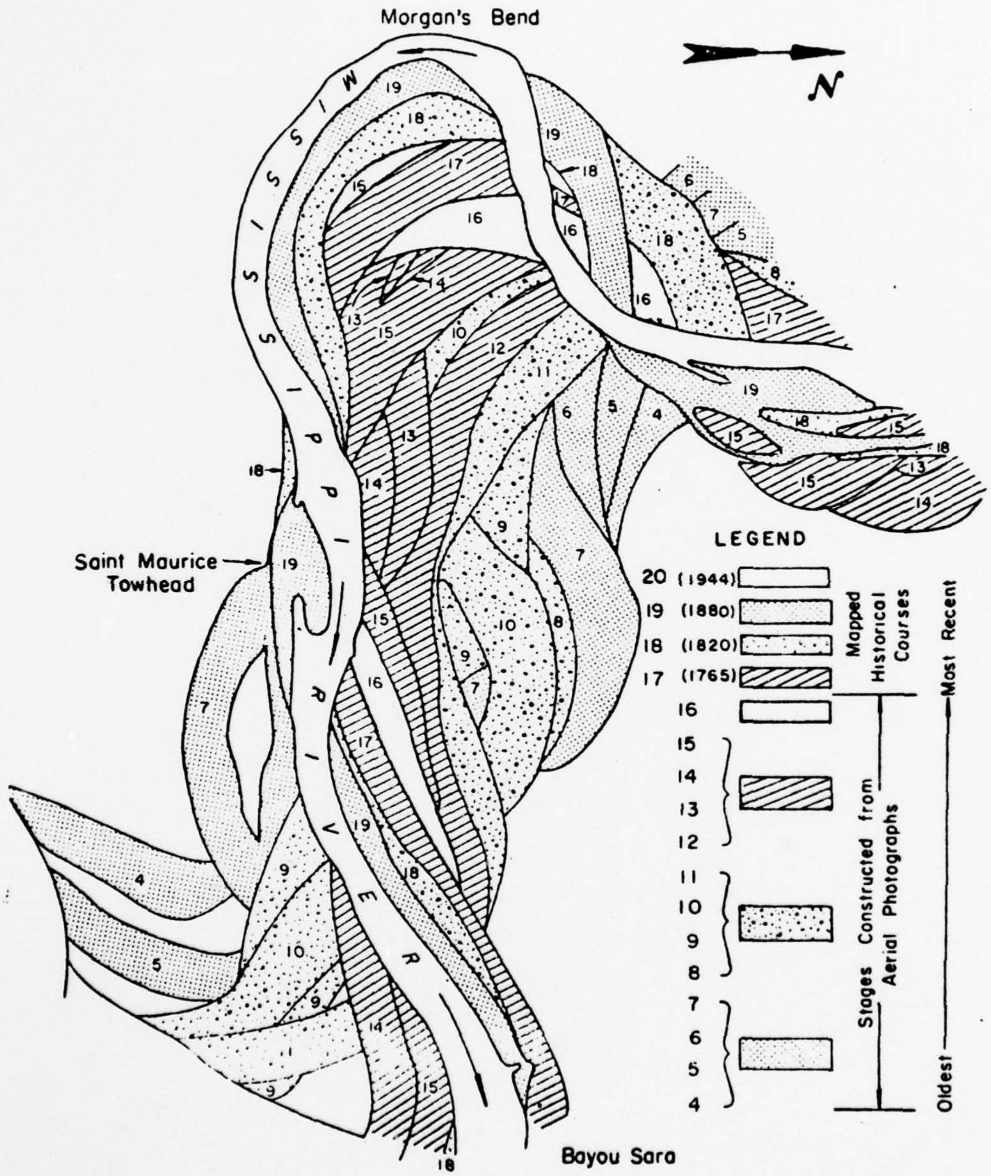


FIGURE 7. Geologic Mississippi River Courses, Vicinity of Morgan's Bend (after Holly et al., 1974, adapted from Fisk).

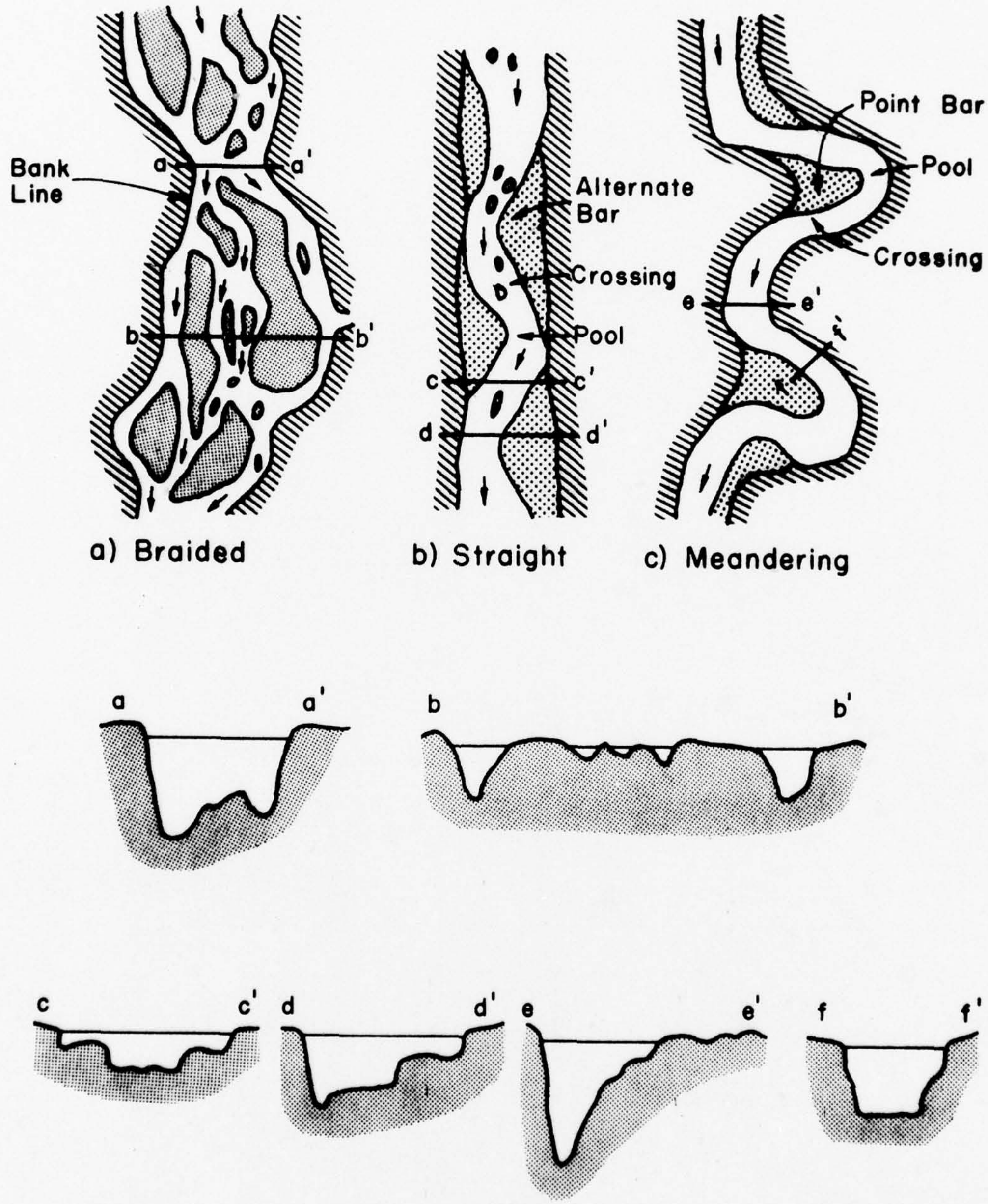


FIGURE 8. River Channel Patterns (after Karaki et al., 1974).

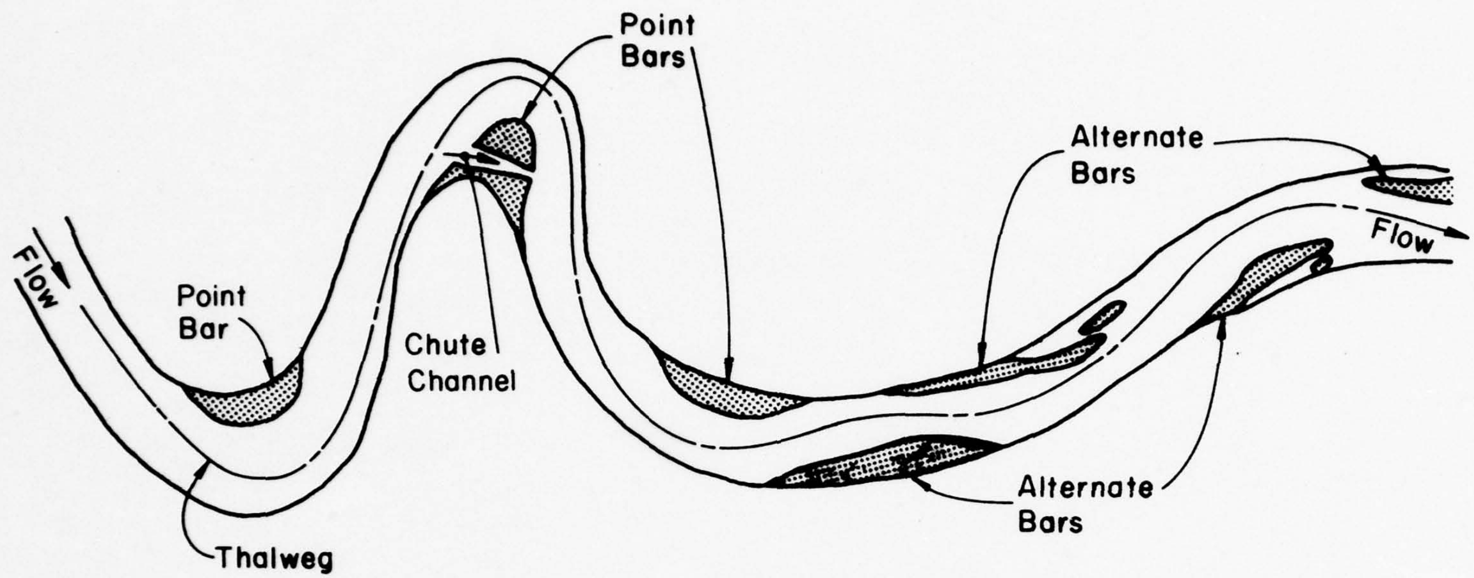


FIGURE 9. Sketch of Meandering and Straight Reaches with Point Bars, Alternate Bars, and Chute Channels (after Cress, 1973).

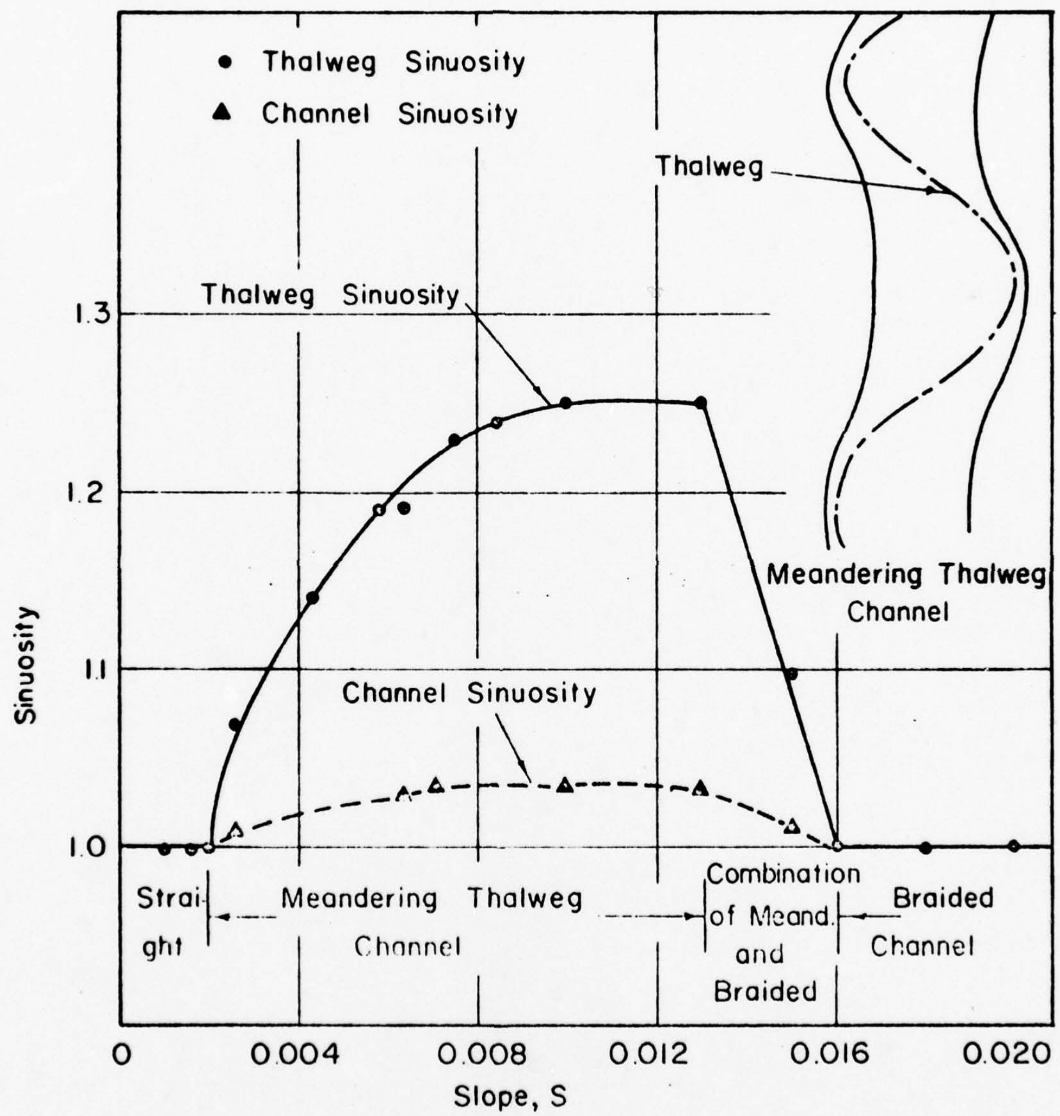
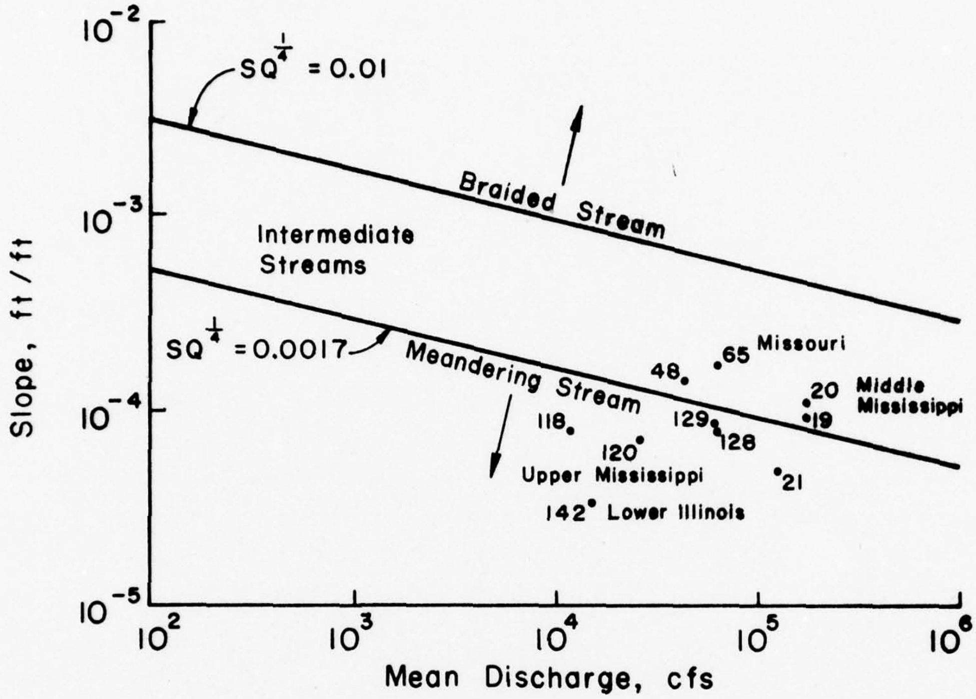


FIGURE 10. Sinuosity versus Slope for a Constant Discharge of 0.15 cfs (after Khan, 1971).



Identification of Reaches Plotted

19 Middle Mississippi - St. Louis to Chester
20 Middle Mississippi - Chester to Cape Giradeau
21 Ohio River
48 Lower Arkansas River
65 Missouri River
118 Upper Mississippi - St. Paul to Redwing
120 Upper Mississippi - La Crosse to Lansing
128 Upper Mississippi - Hannibal to Louisiana
129 Upper Mississippi - Louisiana to Grafton
142 Lower Illinois River

FIGURE 11. Slope-Discharge Relation for Braiding or Meandering in Sand Bed Streams (after Lane, 1957).

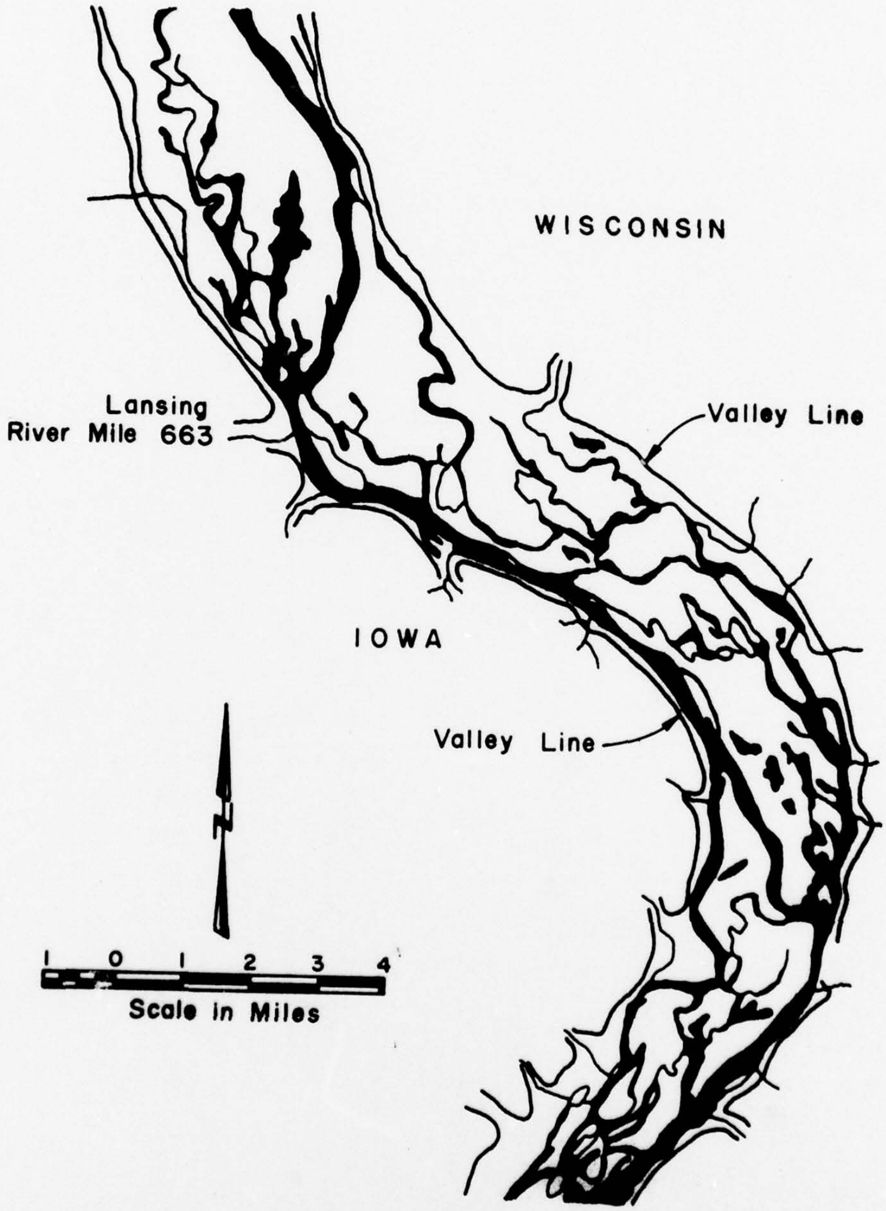
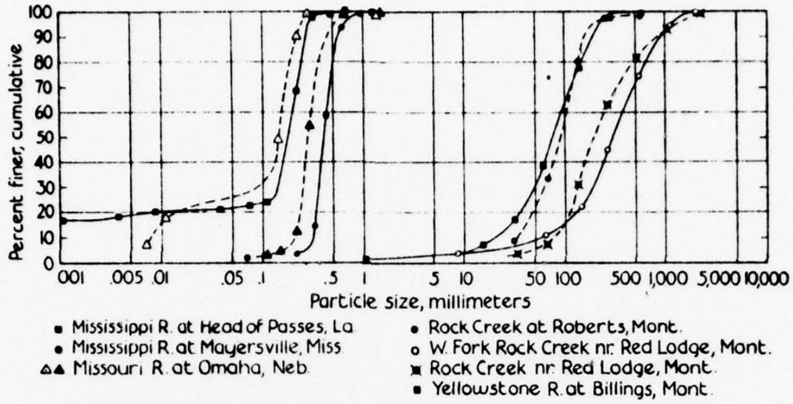
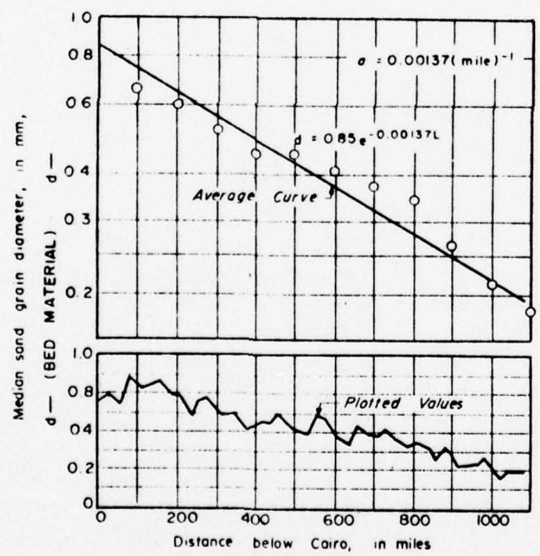


FIGURE 12. A Low-Slope Braided Section of the Upper Mississippi River near Lansing, Iowa (after Lane, 1957).



(a) Samples of Size Distribution of Bed Material, Yellowstone-Missouri-Mississippi River System (after Leopold et al., 1964).



(b) Particle size reduction along the Mississippi River (after Fenwick, 1969).

FIGURE 13. Particle Size Distribution--Mississippi River.

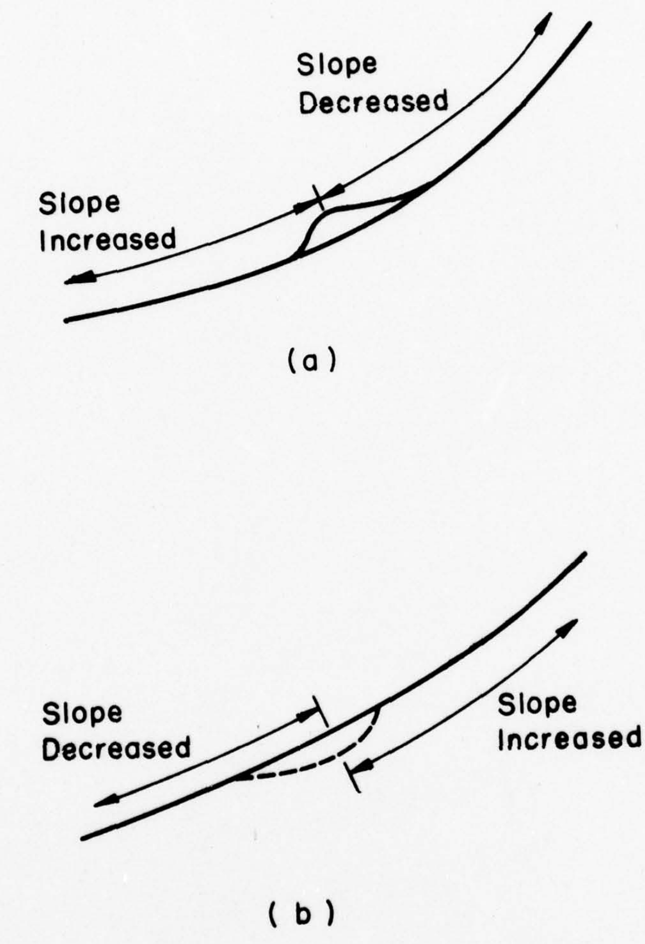


FIGURE 14. Adjustments in Longitudinal Profile
(after Morisawa, 1968).

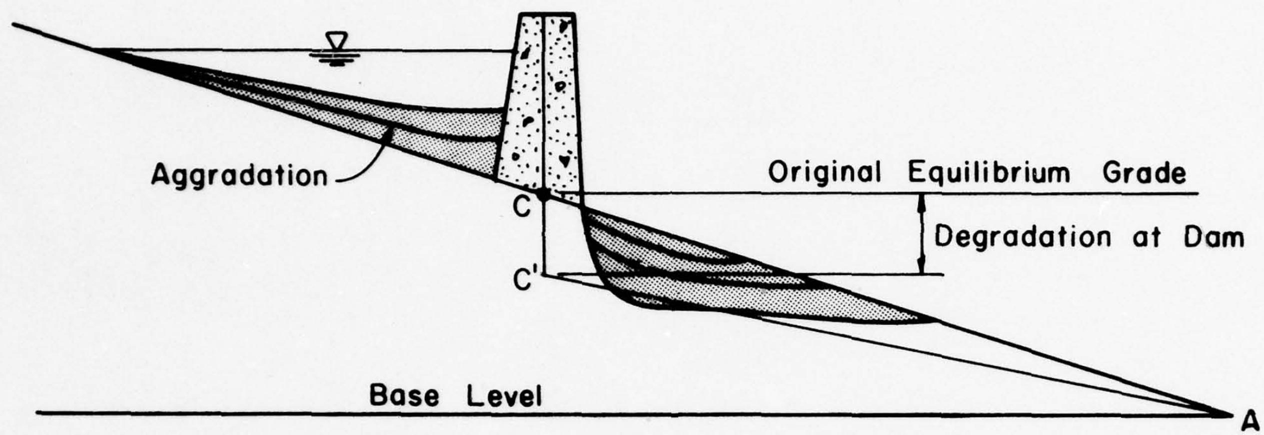


FIGURE 15. Channel Adjustment Above and Below a Dam.

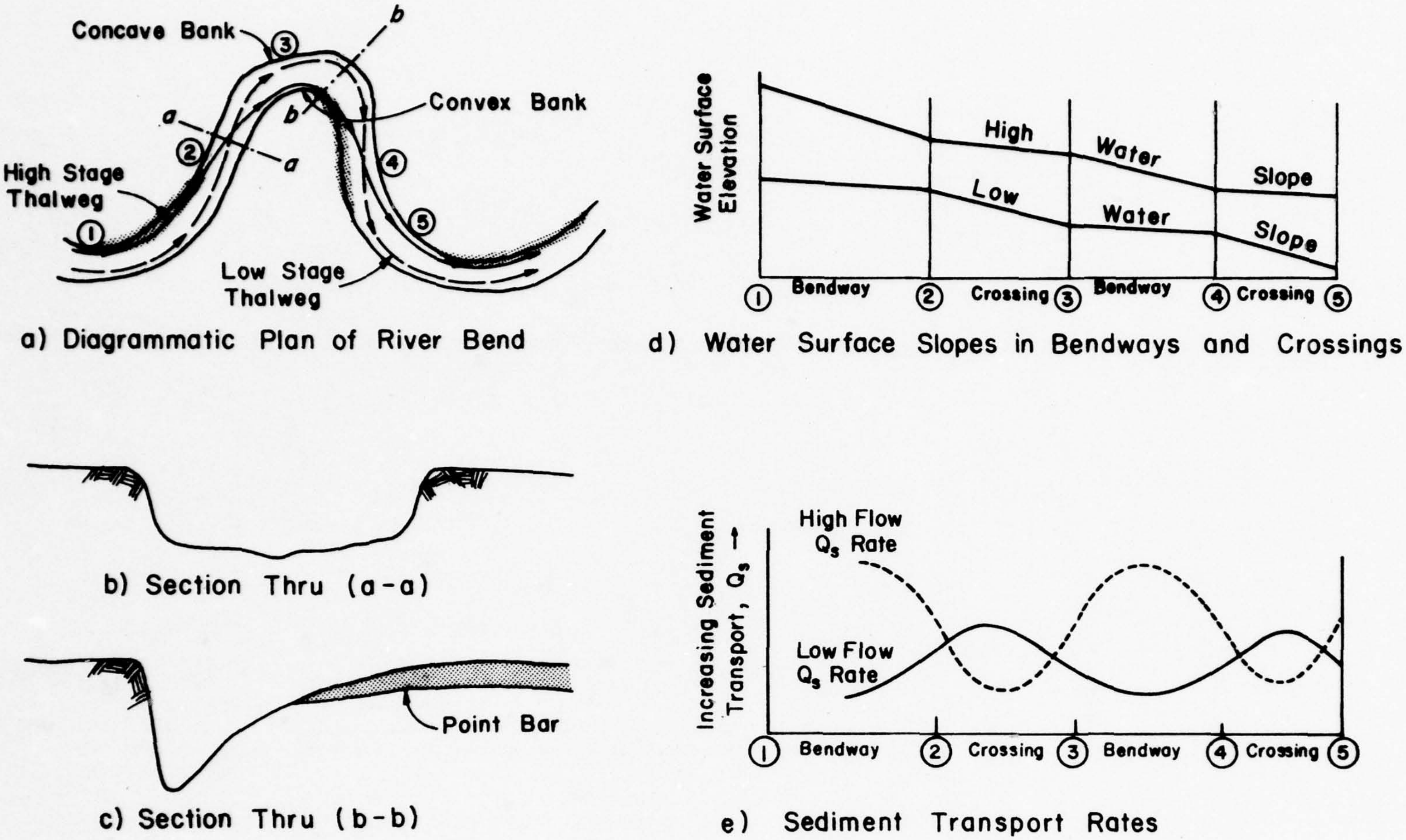


FIGURE 16. Characteristics of a River Bendway (after Fenwick, 1969).

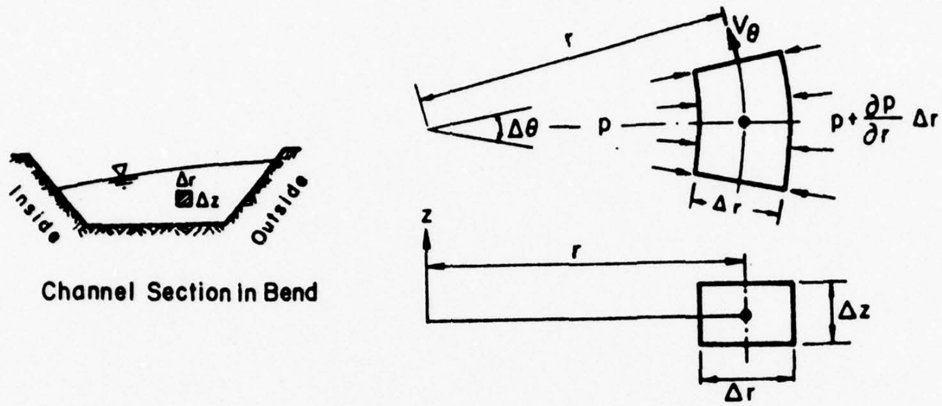


FIGURE 17. Definition Sketch of Dynamics of Flow Around a Bend.

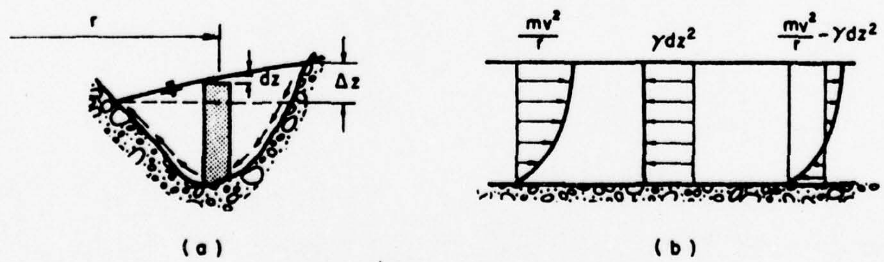
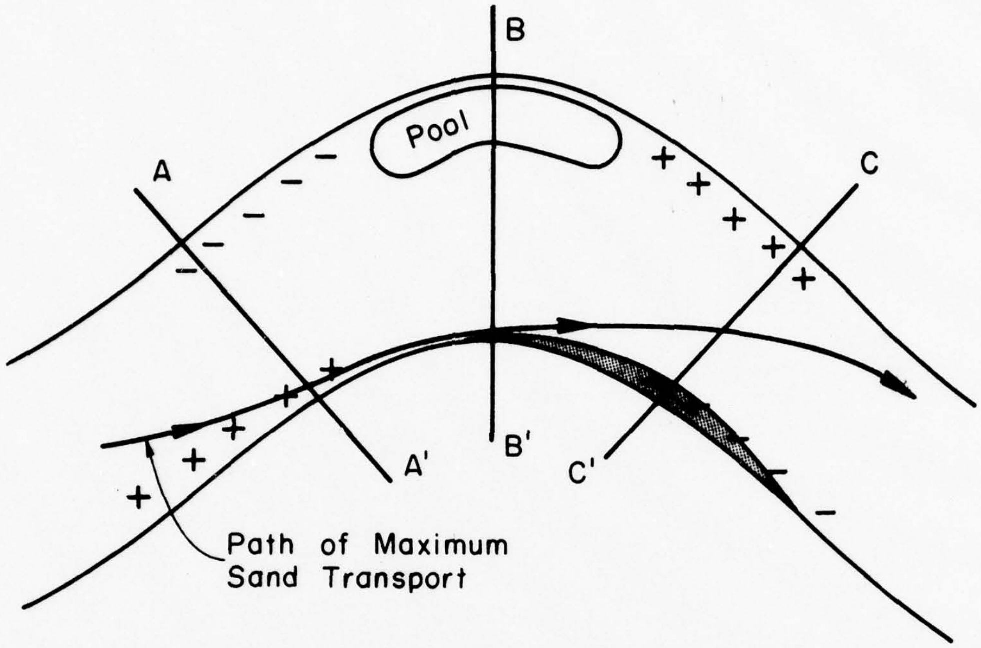


FIGURE 18. Schematic Representation of Transverse Currents in a Channel Bend.

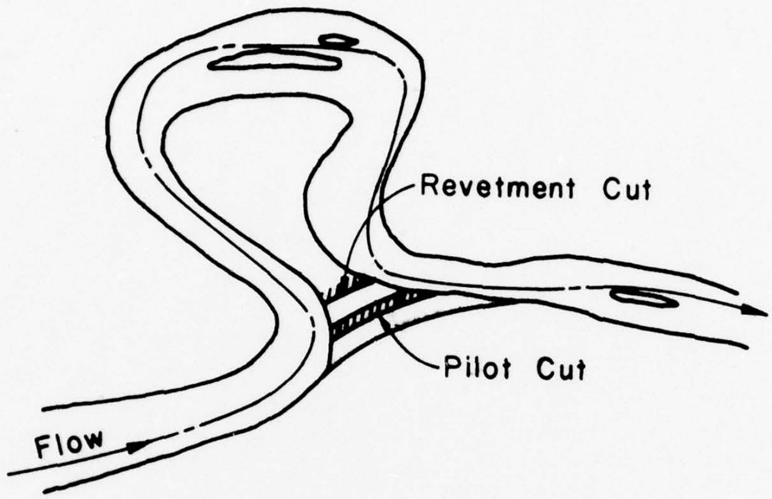


a) Zones of Increasing (+) and Decreasing (−) Bed Shear Stress, and Sand Transport.

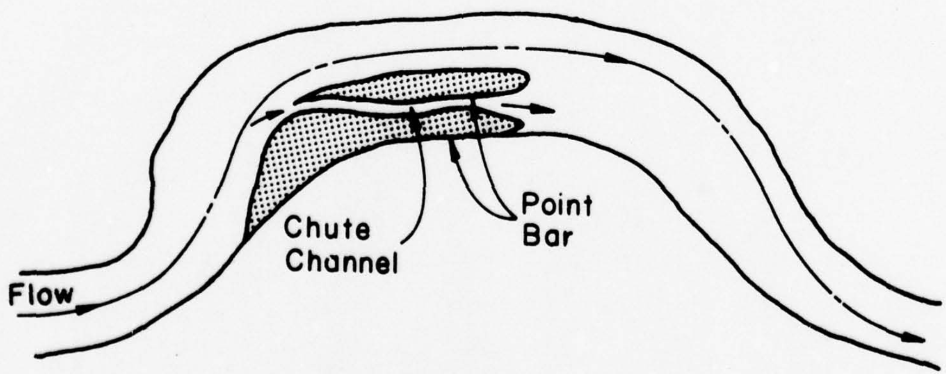
Bank	Supply	− Shear	= Net
Concave			
AB	−	−	O
BC	−	+	−
Convex			
A'B'	+	+	O
B'C'	+	−	+

b) Tabulated Regions of Deposition (+), Erosion (−) and Dynamic Equilibrium (O).

FIGURE 19. Influence of Secondary Currents, Shear Stress, and Sediment Distribution on Bendway Morphology.



a) Natural or Dredged Cutoff



b) Chute Channel

FIGURE 20. Island Formation by Cutoff of Meander Loop and Chute Channel Development (after Cress, 1973).

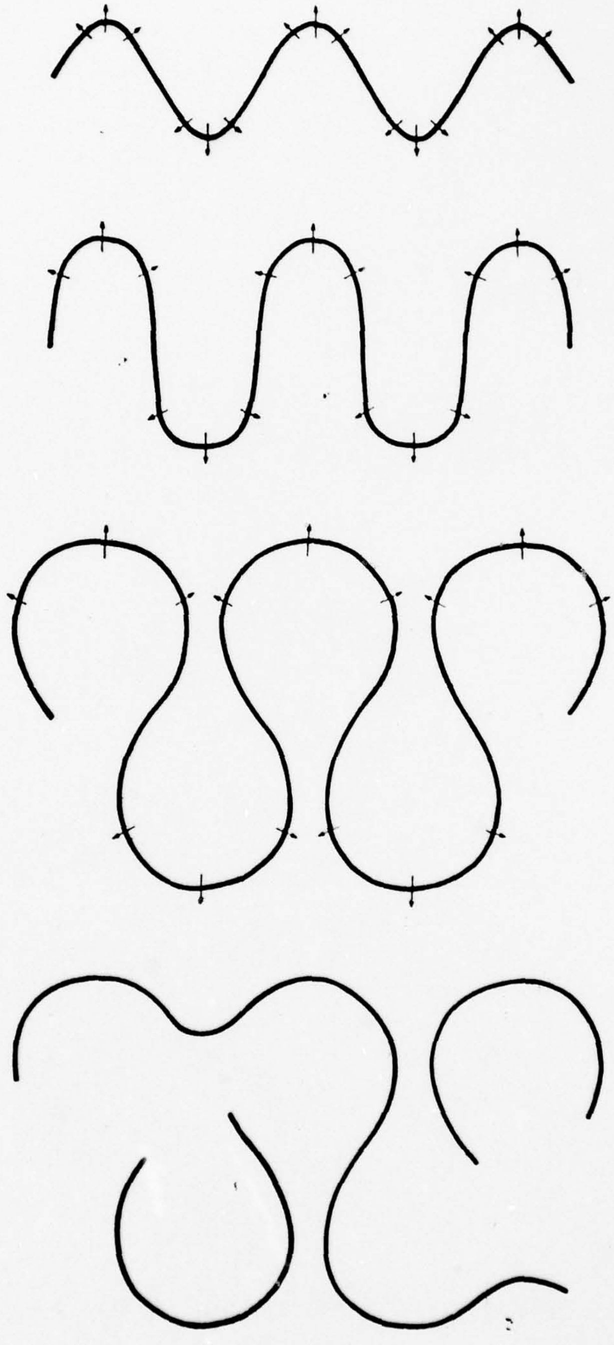
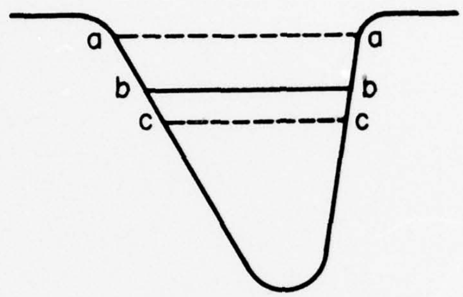
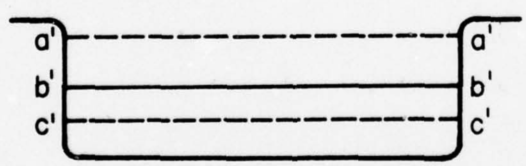


FIGURE 21. Development of Natural Cutoffs (after NEDECO, 1959).



a) At a Bend



b) At a Crossing

FIGURE 22. Variation in Stage at Typical Cross Sections (after Lane and Borland, 1954).

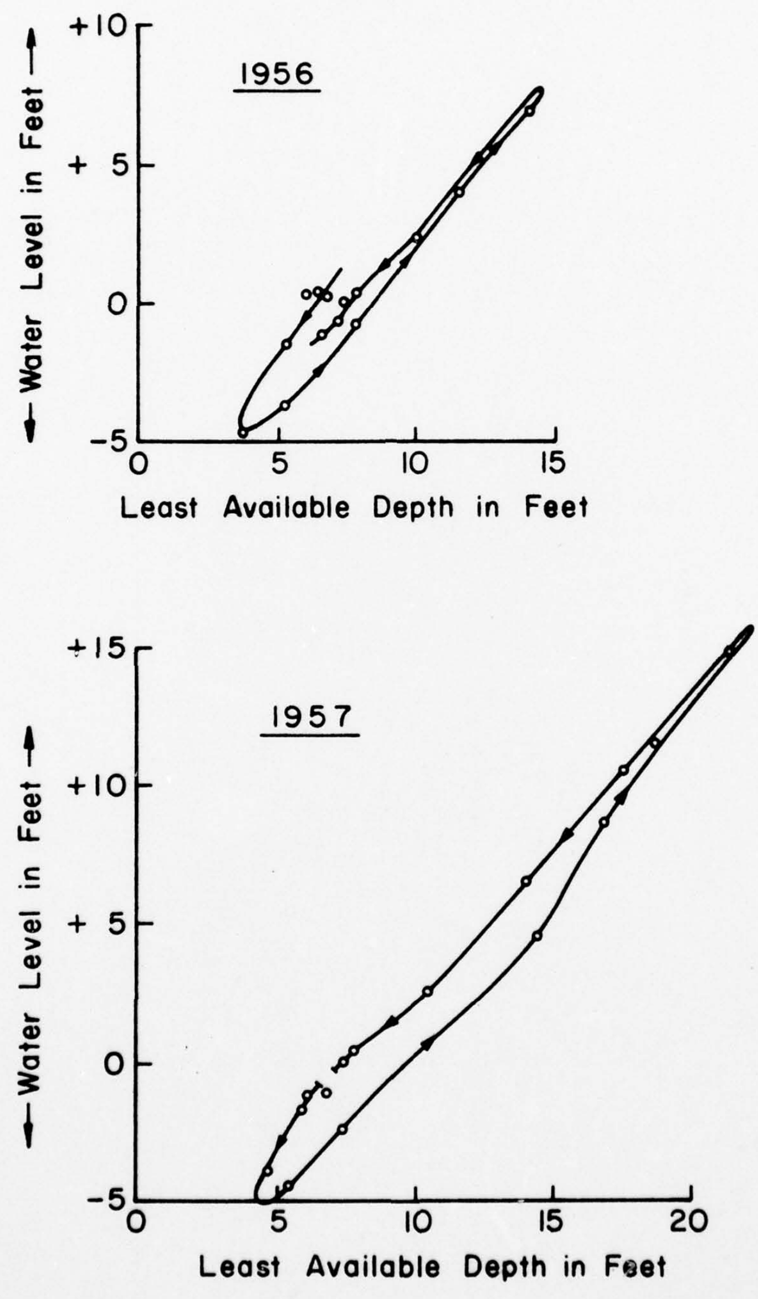


FIGURE 23. Available Depth Versus Water Level at a Crossing, Niger River, Nigeria (after NEDECO, 1959).

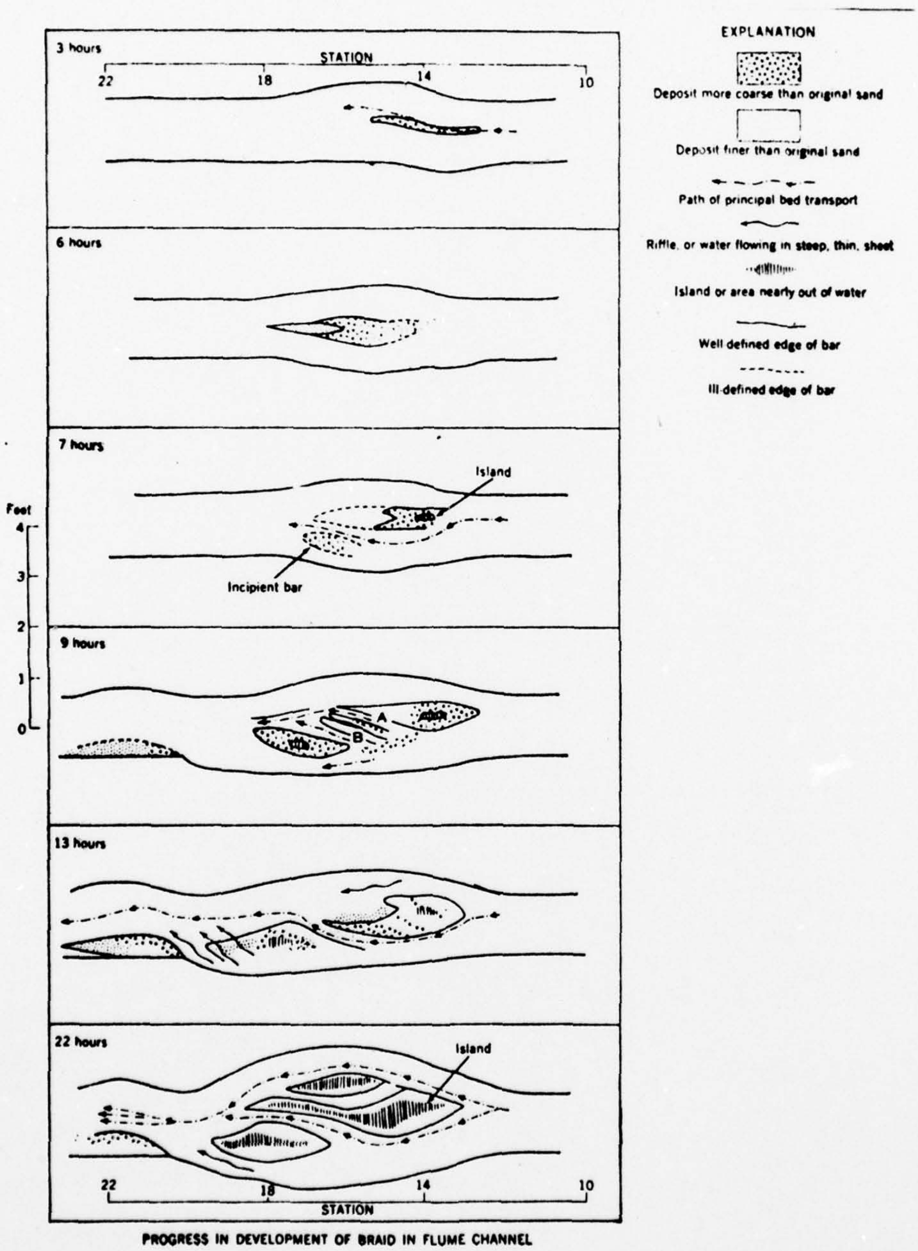


FIGURE 24. Island and Chute Channel Evolution in a Flume (after Leopold and Wolman, 1957).

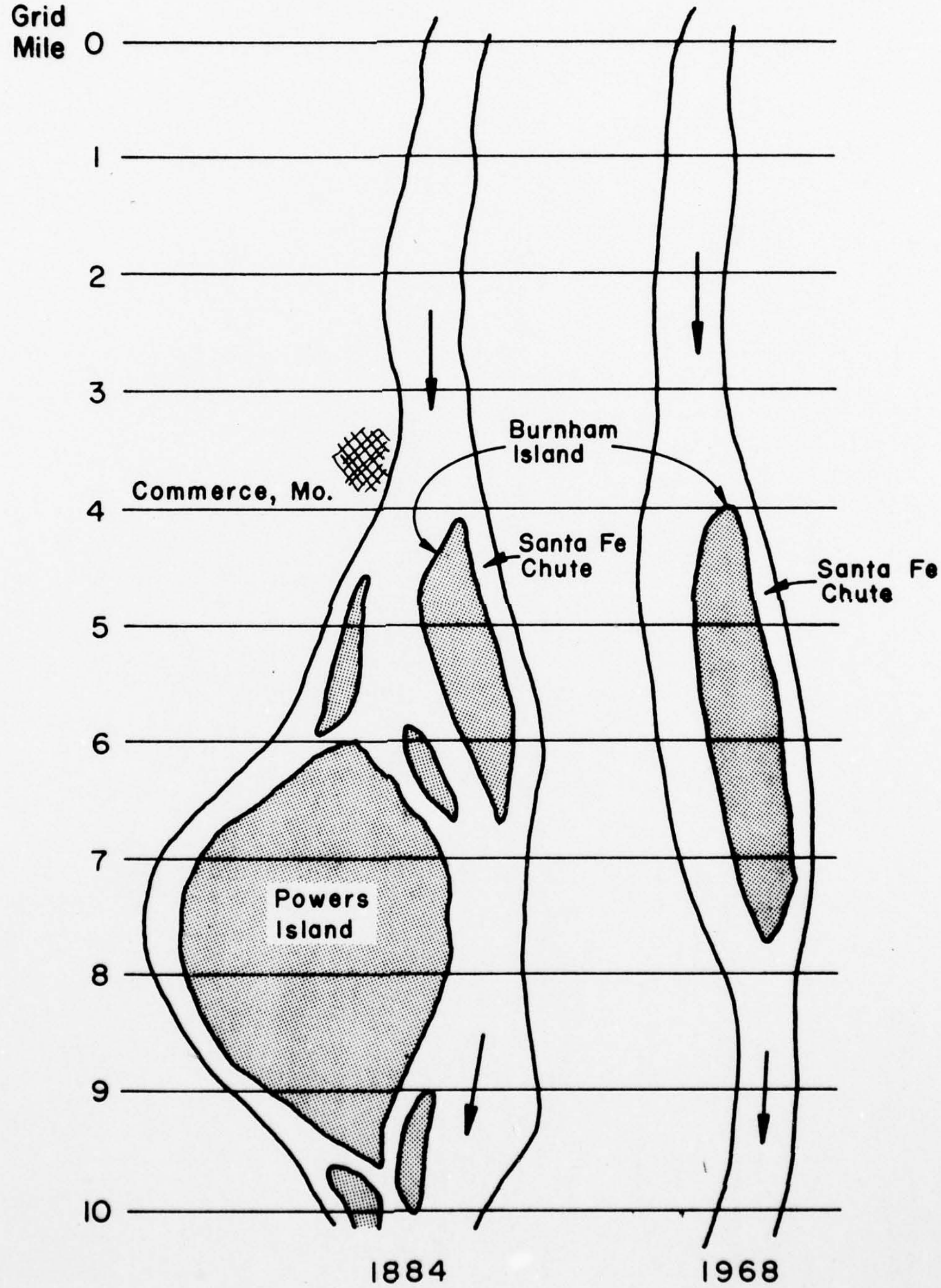


FIGURE 25. Loss of Chute Channels below Thebes Gap (after Simons et al., 1974).

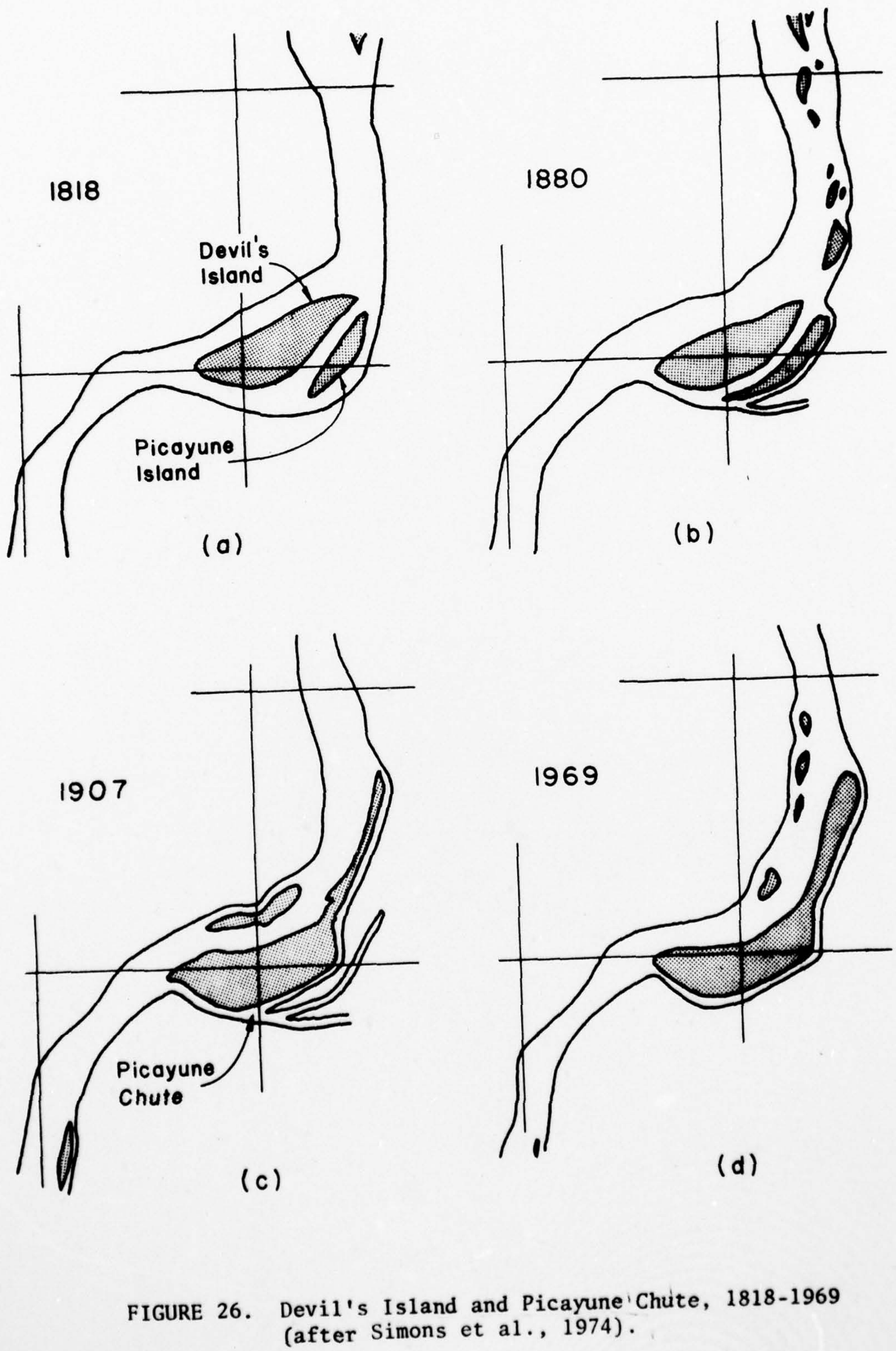


FIGURE 26. Devil's Island and Picayune Chute, 1818-1969 (after Simons et al., 1974).

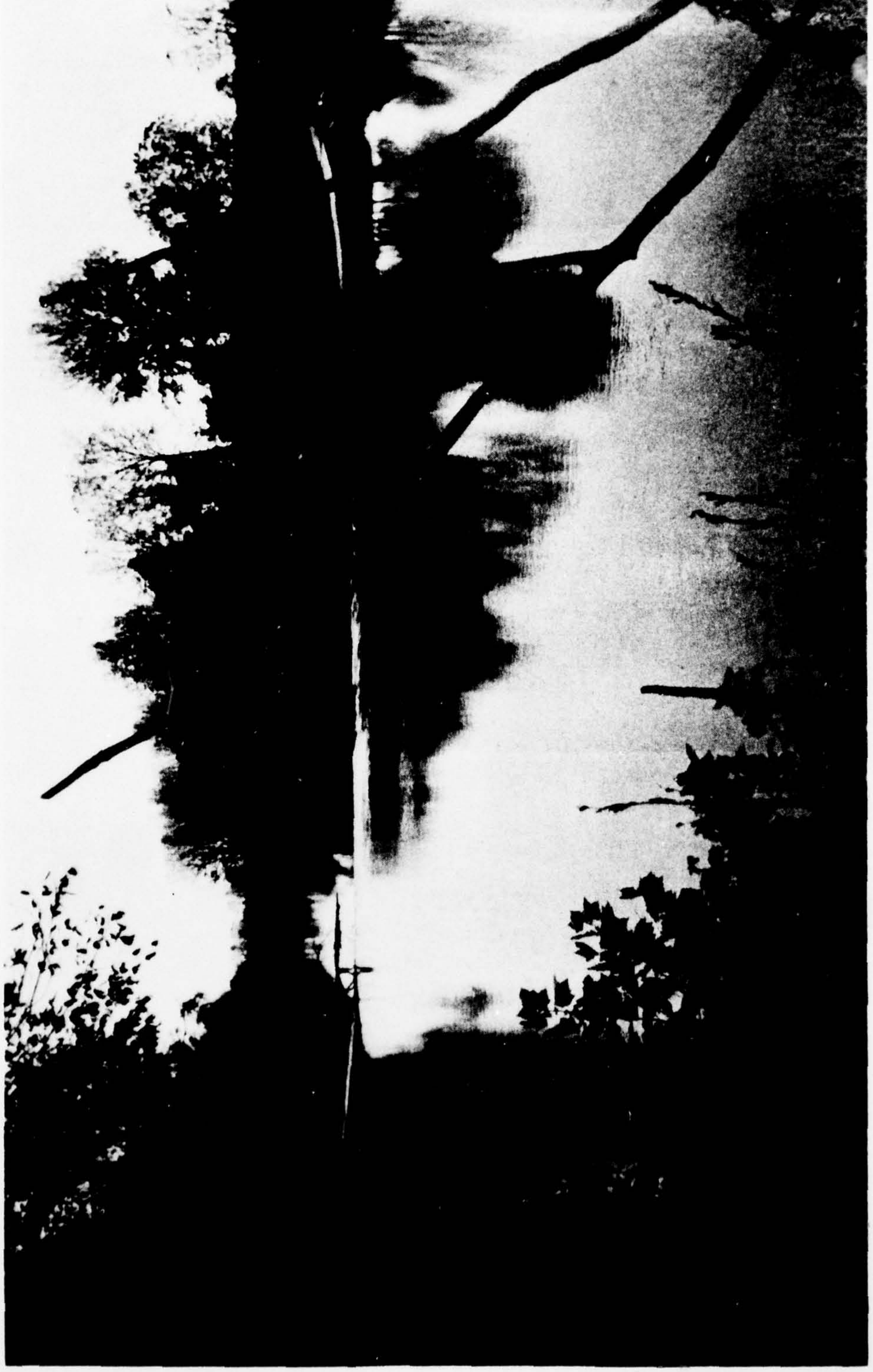
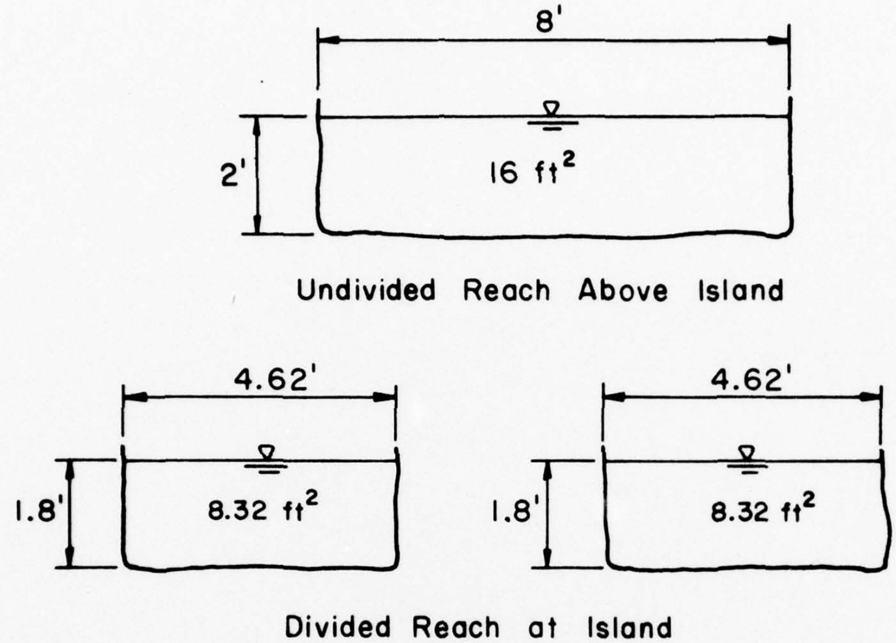


FIGURE 27. Picayune Chute in 1972.

FIGURE 28. Dike Fields in the Mississippi River.



	Undivided	Divided	% Change
Area (A)	16 ft^2	16.64	+ 4
Width (W)	8 ft	9.28	+16
Average Depth (D)	2 ft	1.8	-10
D/W to T	.25	.19	-22
D/W Branch	.25	.39	+55
Wetted Perimeter (P)	12 ft	16.44	+37
Hydraulic Radius ($\frac{A}{P}$)	1.33 ft	1.01	-24
Average Velocity (V)			- 4
Slope (S)			+10

FIGURE 29. Change in Channel Characteristics from Undivided to Divided Reach Based on Rubey's (1952) Data.

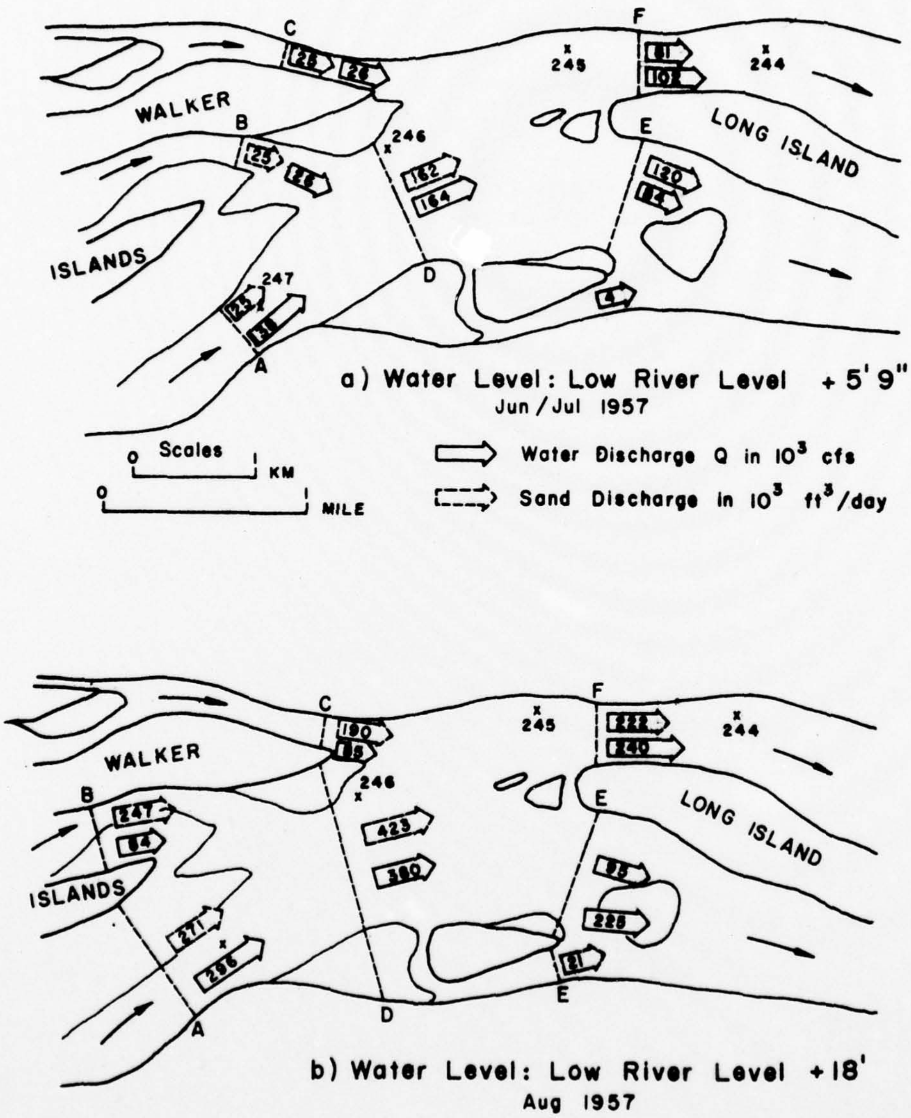


FIGURE 30. Sand Transport and Water Discharge near Long Island, Niger River, Nigeria (after NEDECO, 1959).

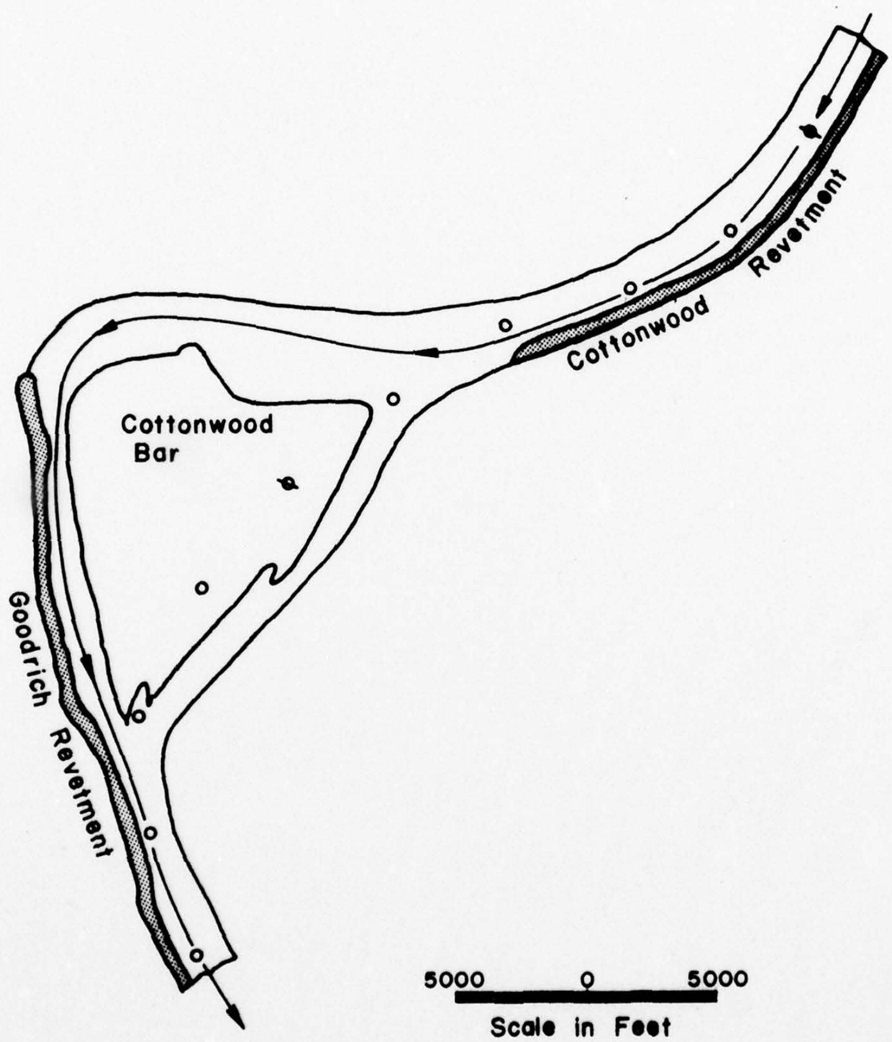


FIGURE 31. Divided Reach--Mississippi River (after Haas, 1963).

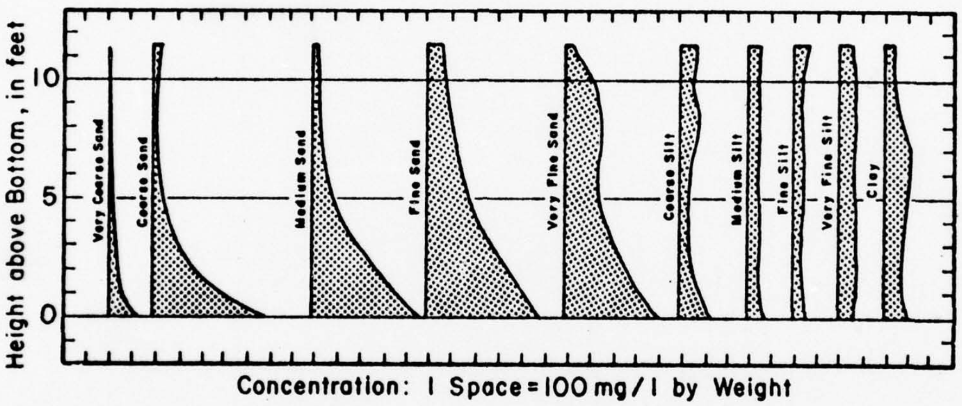


FIGURE 32. Discharge Weighted Concentration of Suspended Sediment for Different Particle Size Groups at a Sampling Vertical in the Missouri River (after Guy, 1970).

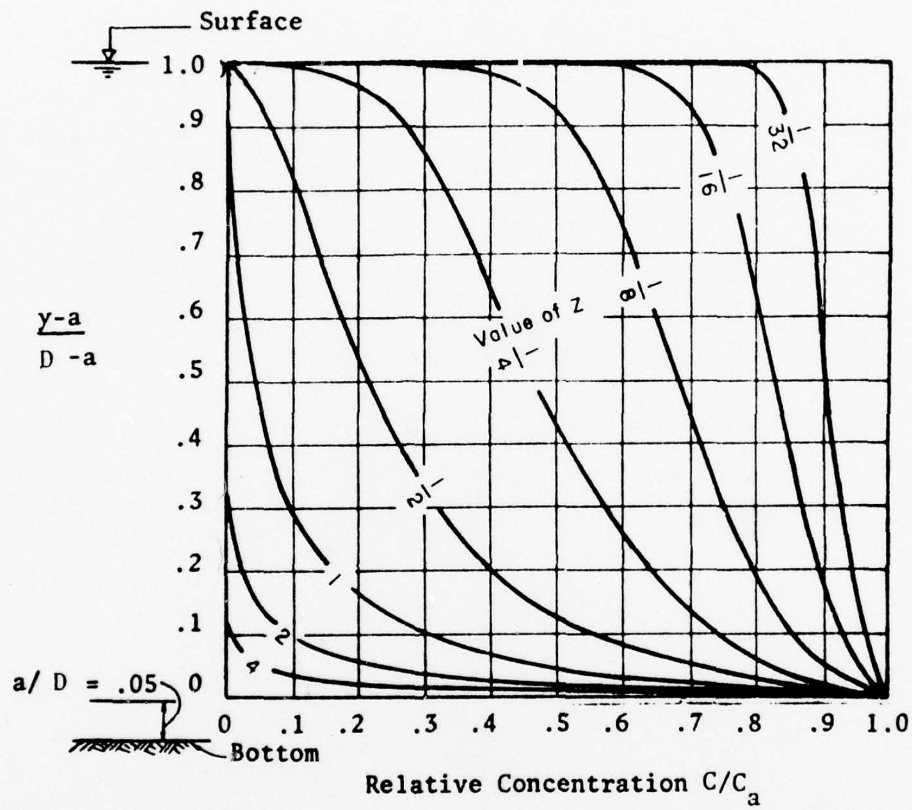


FIGURE 33. Graph of Suspended Sediment Distribution (after Melone, 1974).

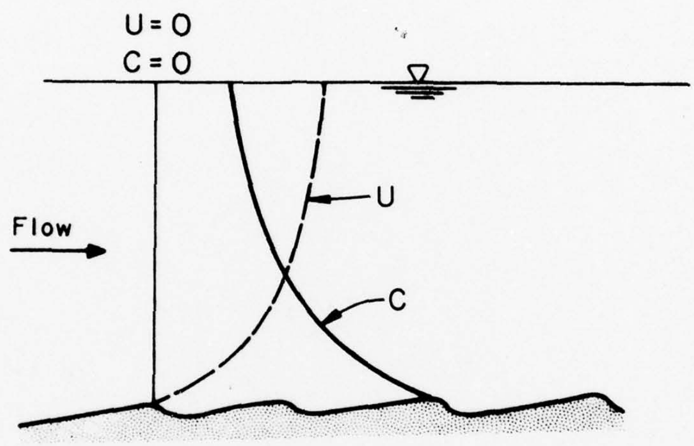


FIGURE 34. Schematic Sediment and Velocity Profiles.

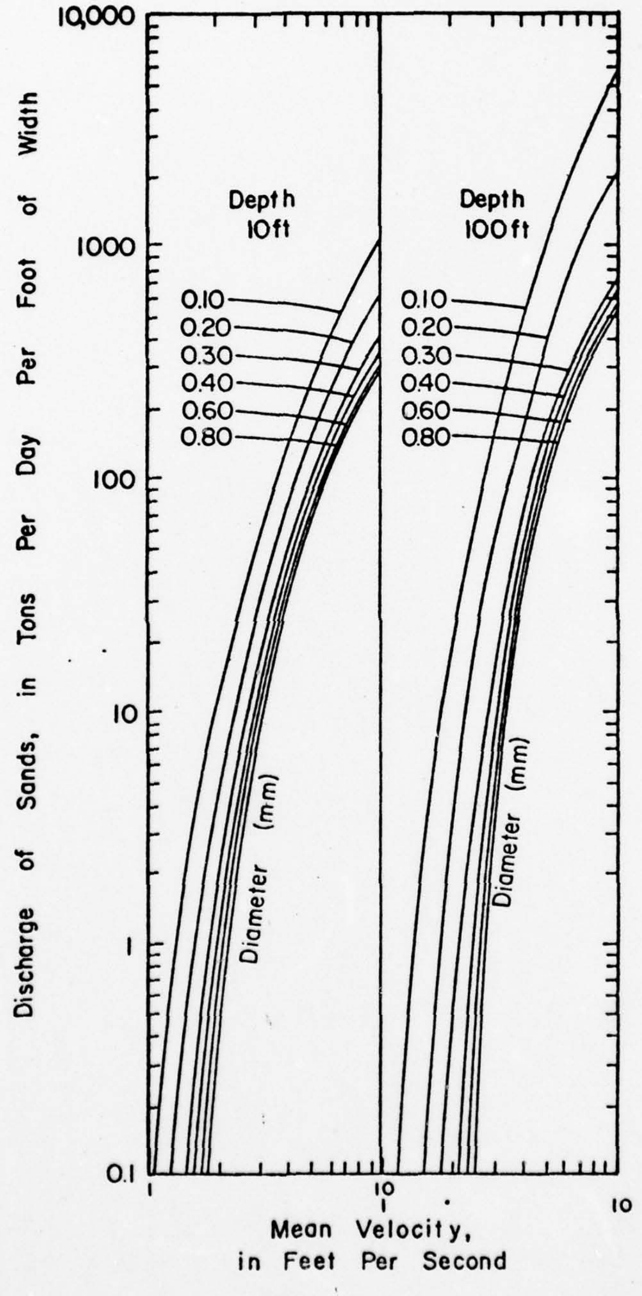


FIGURE 35. Relation of Discharge of Sands to Mean Velocity for Six Median Sizes of Bed Sands, Depth of Flow, and a Water Temperature of 60° F (Colby, 1964).

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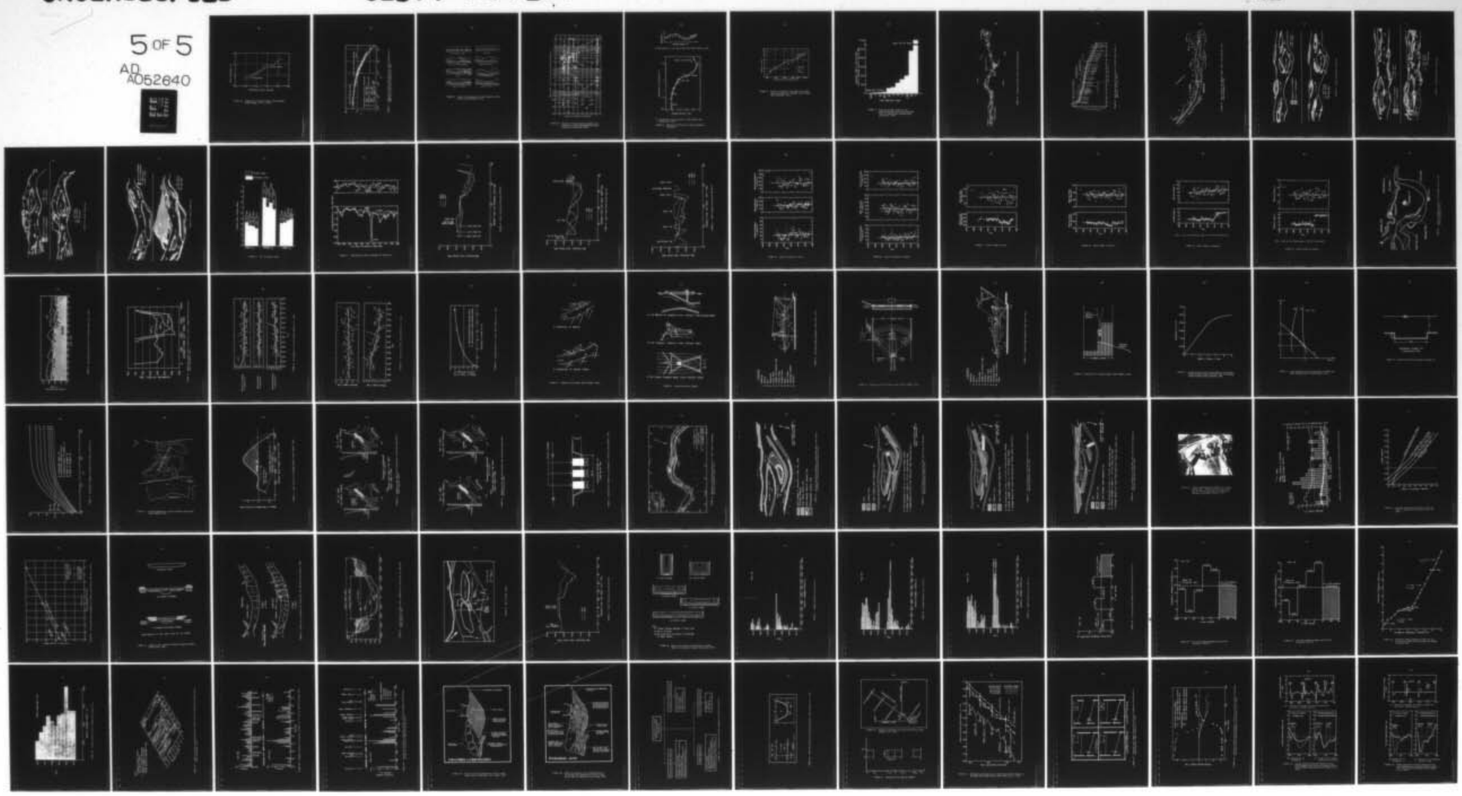

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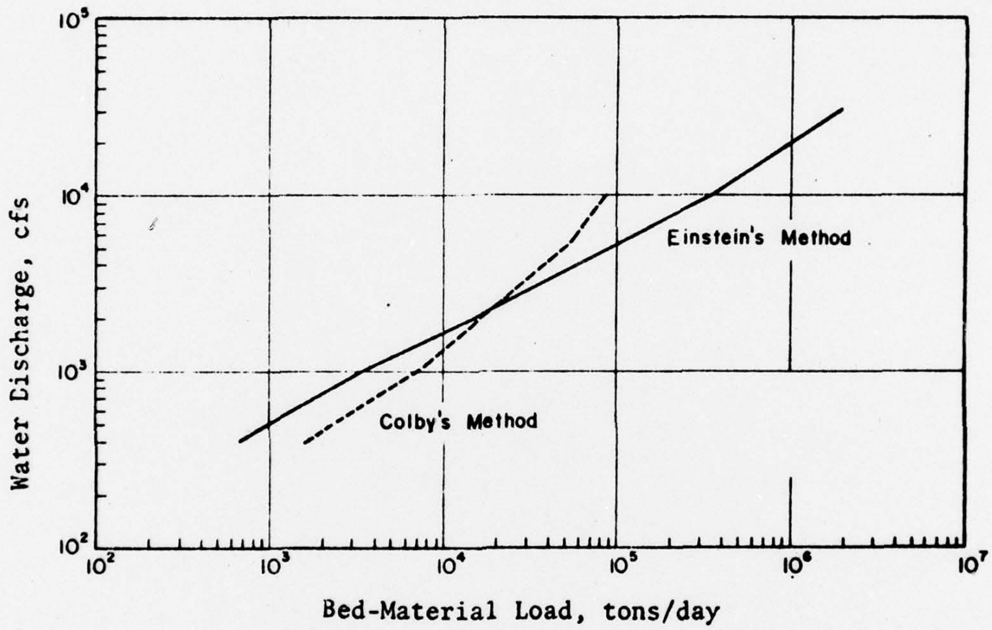


FIGURE 36. Comparison of Einstein versus Colby Methods (after Karaki, et al., 1973).

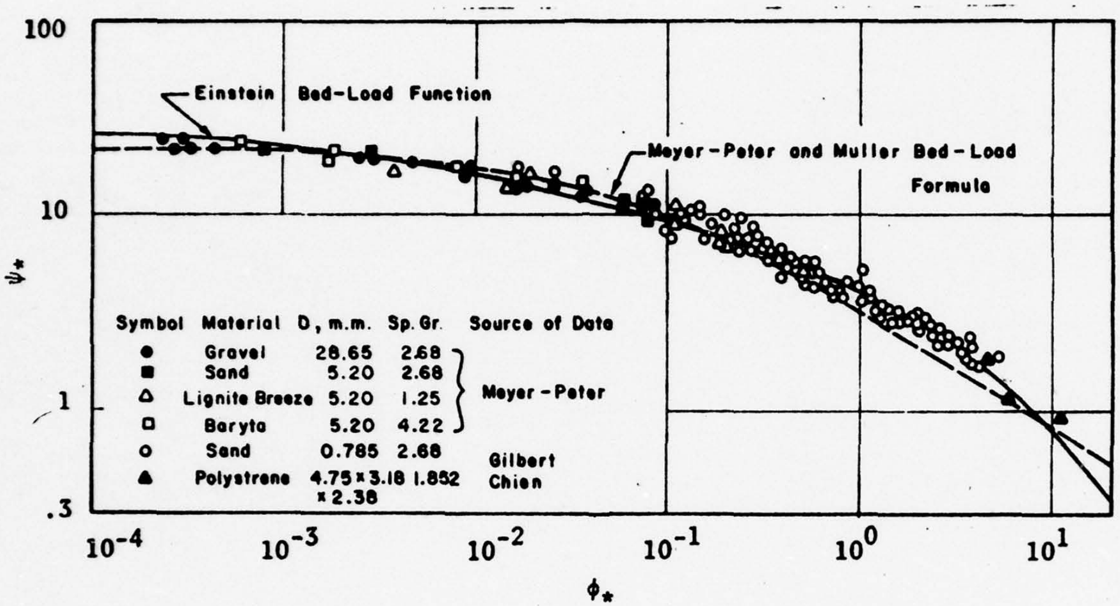


FIGURE 37. Comparison of Meyer-Peter Muller and Einstein Methods for Computing Contact Load (after Chien, 1954).

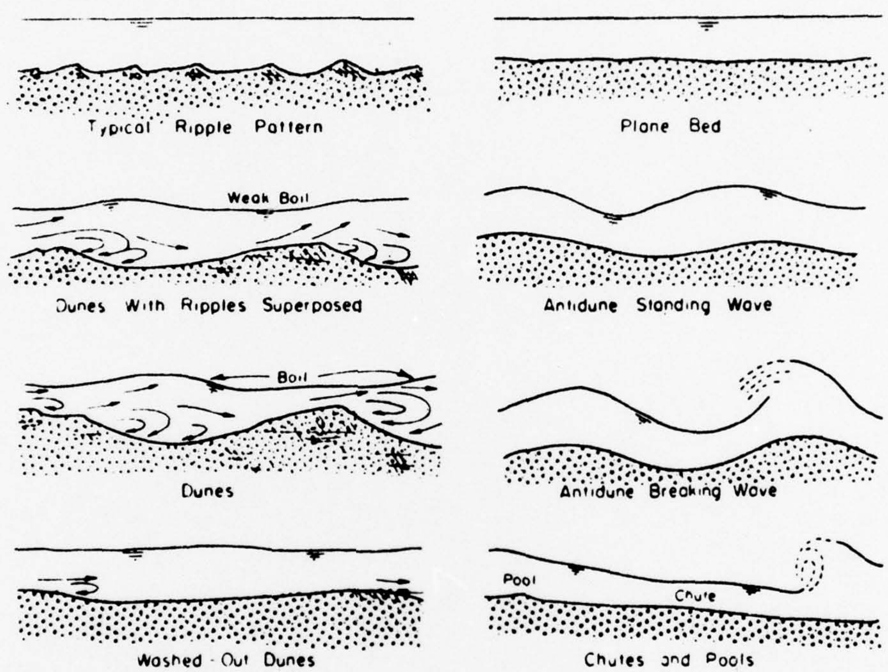


FIGURE 38. Forms of Bed Roughness in Sand Channels (after Simons and Richardson, 1971).

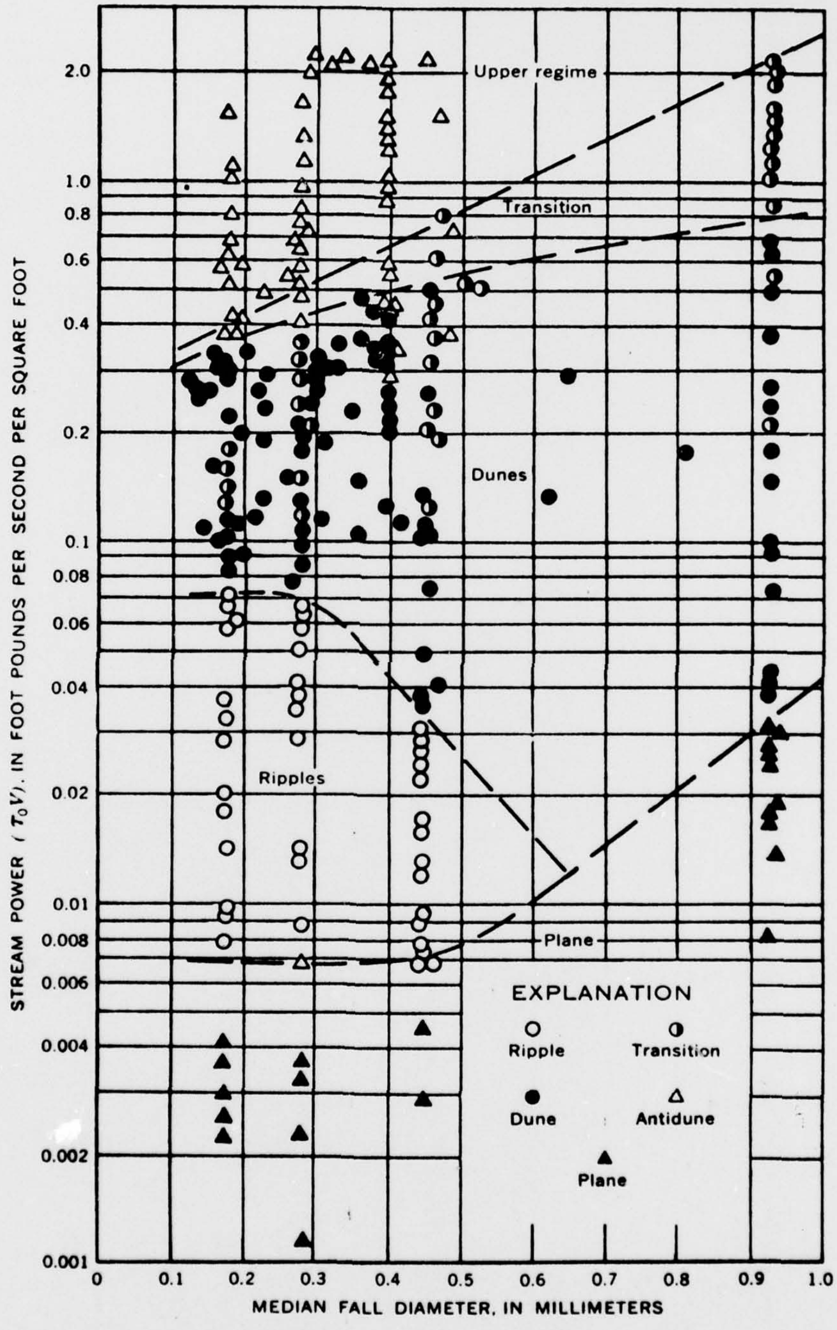
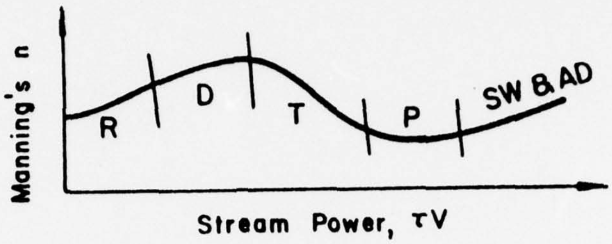
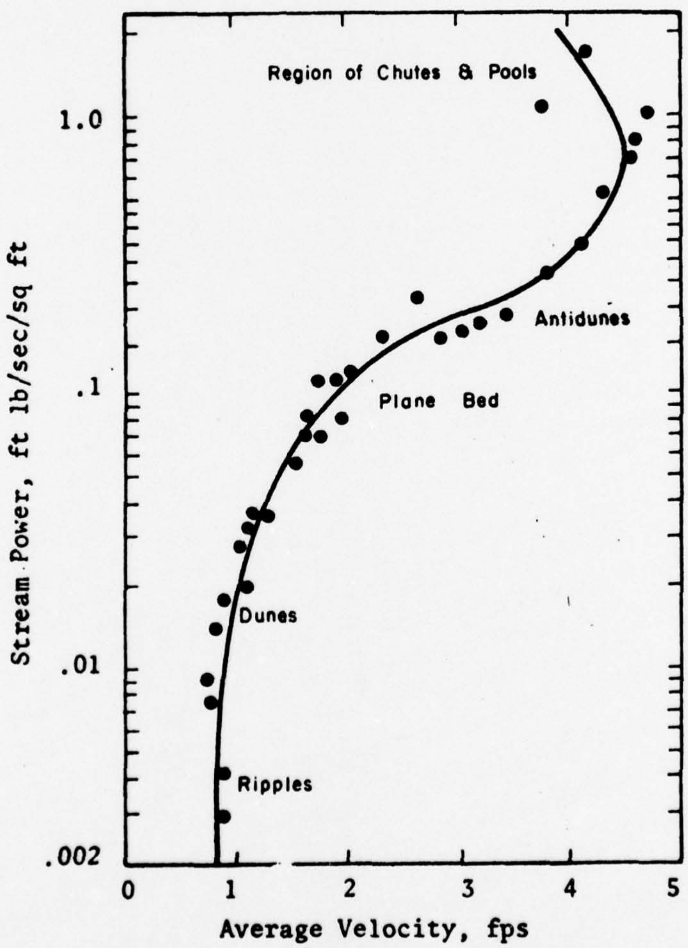


FIGURE 39. Relation of Stream Power and Median Fall Diameter to Form of Bed Roughness (after Simons and Richardson, 1966).



(a) Resistance to Flow versus Bed Form (after Melone, 1974).



(b) Stream Power versus Velocity (after Simons and Richardson, 1966).

FIGURE 40. Relation of Bed Form to Various Hydraulic Parameters.

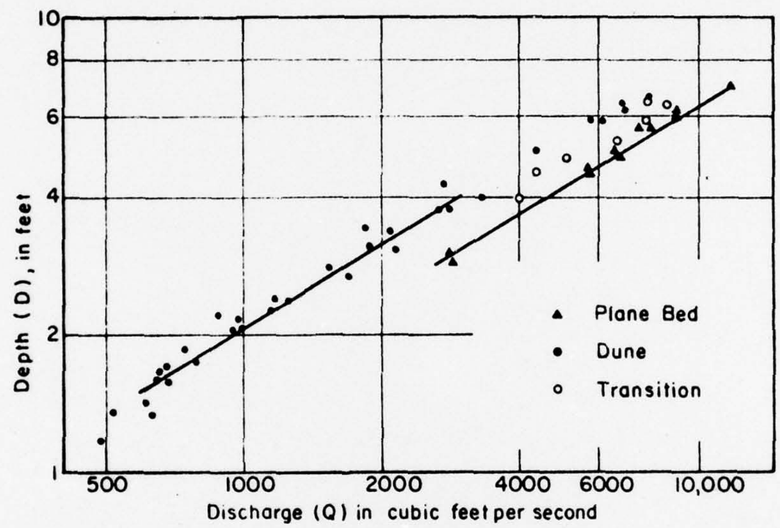


FIGURE 41. Relation of Depth to Discharge for Elkhorn River near Waterloo, Nebraska (after Simons and Richardson, 1971).

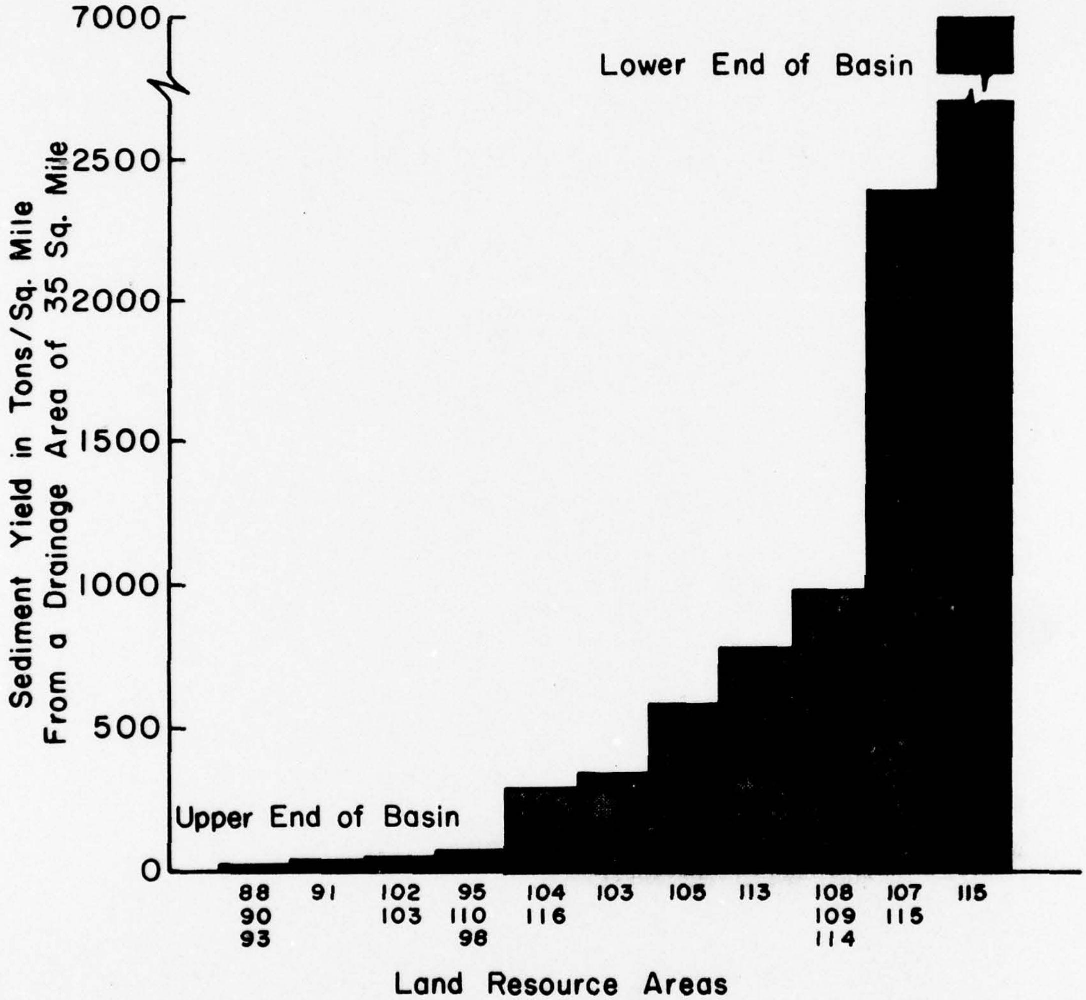


FIGURE 42. Relative Sediment Yields in the Upper Mississippi River Basin Expected from a 35 Square Mile Drainage Area (after Mack, 1970).

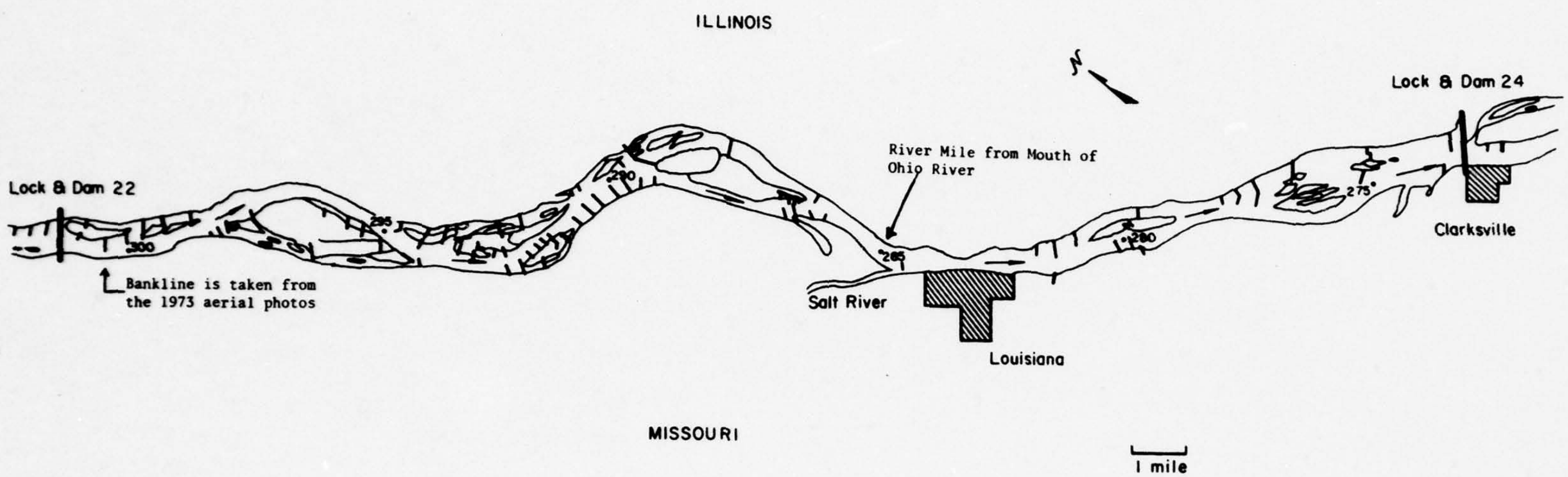


FIGURE 43. Location of Dikes in Pool 24, Mississippi River.

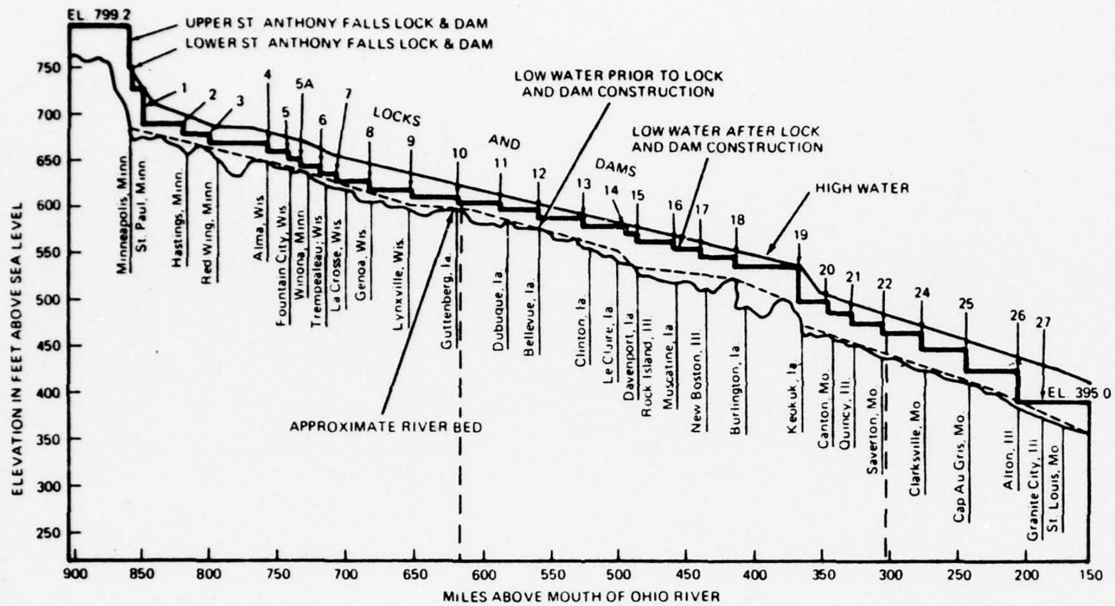


FIGURE 44. Upper Mississippi River Basin Mile 198-858 (after Corps of Engineers, St. Paul, 1974).

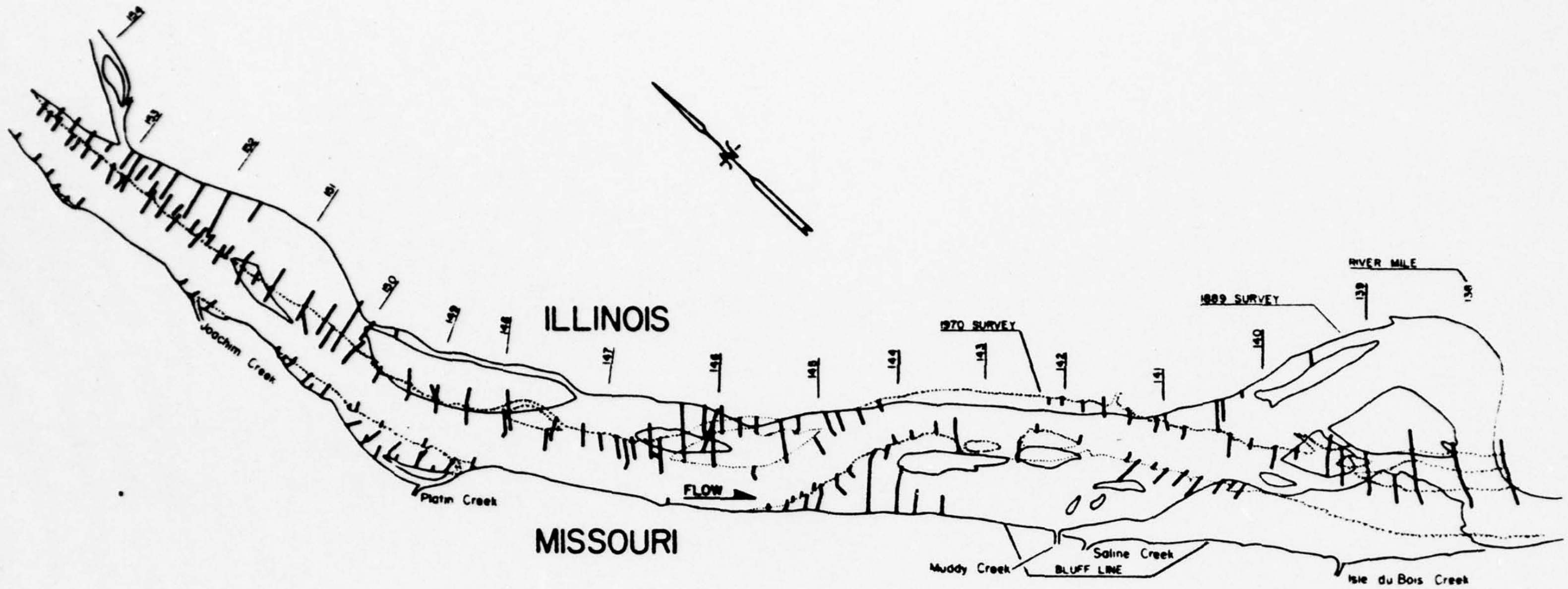


FIGURE 45. Dike Locations and Channel Change, 1889 to 1970, RM 154 to 138 (after Degenhardt, 1973).

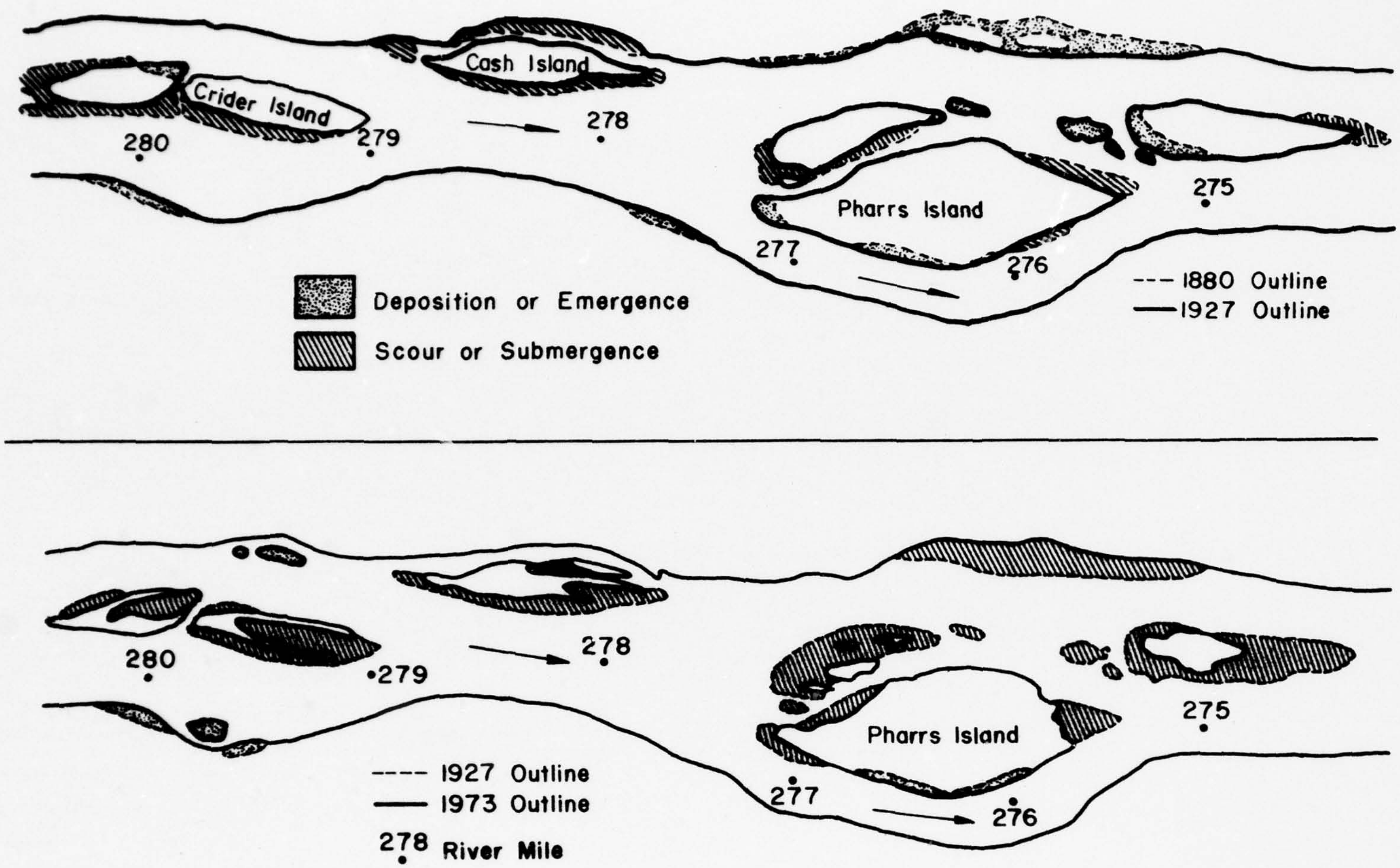


FIGURE 46. Crider-Cash-Pharrs Island Area.

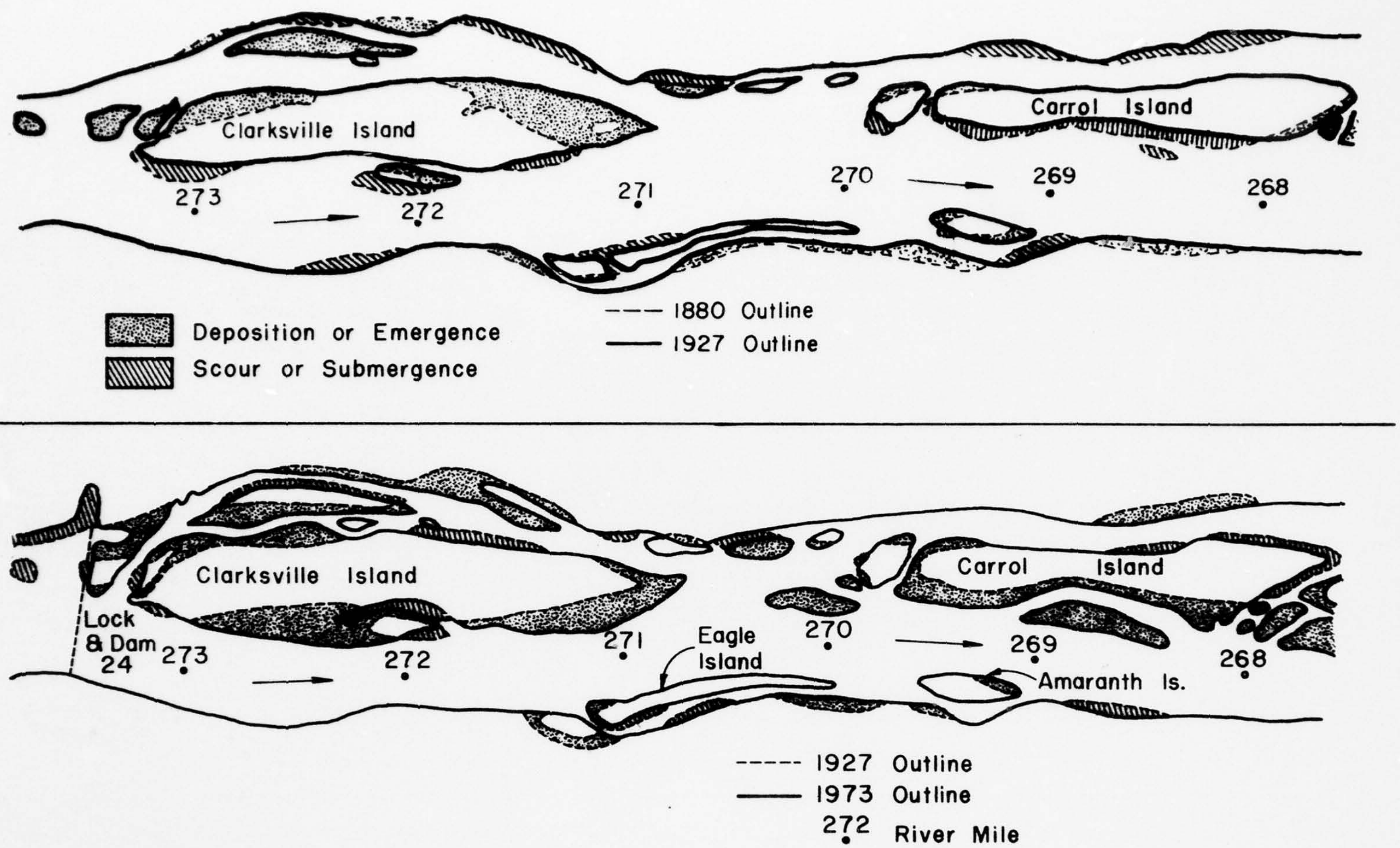


FIGURE 47. Clarksville-Carrol Island Area.

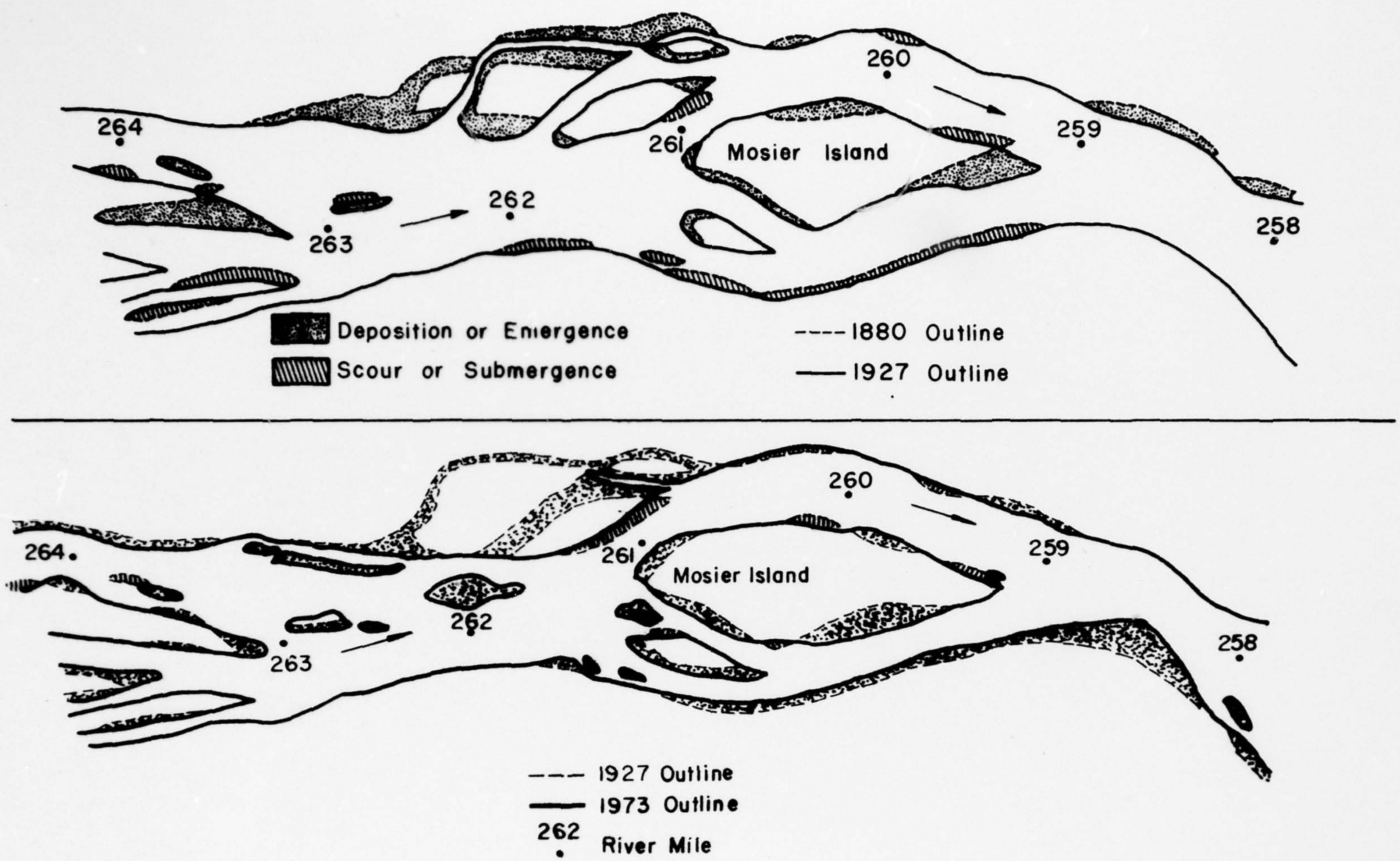


FIGURE 48. Mosier Island Area.

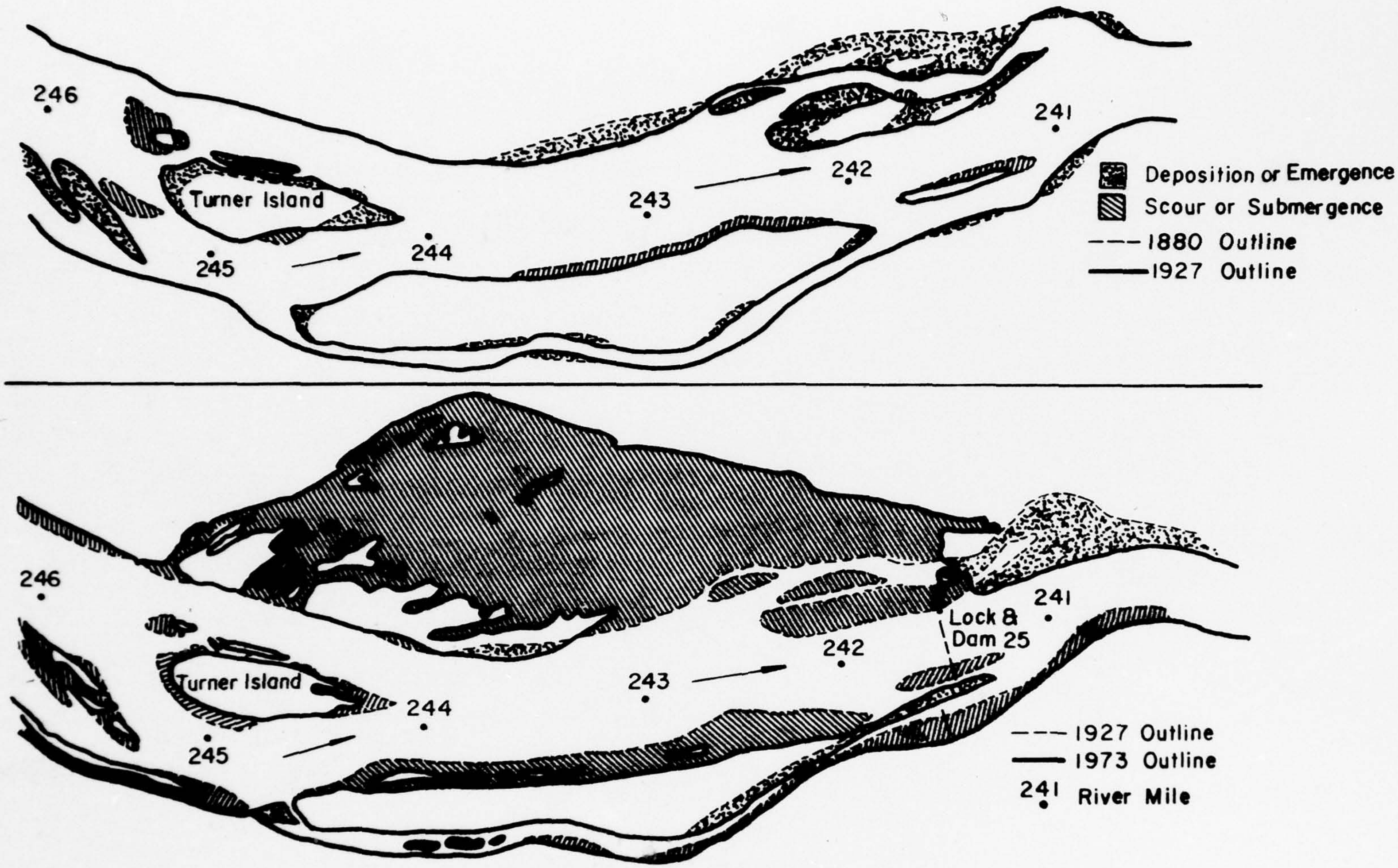


FIGURE 49. Turner Island Area.

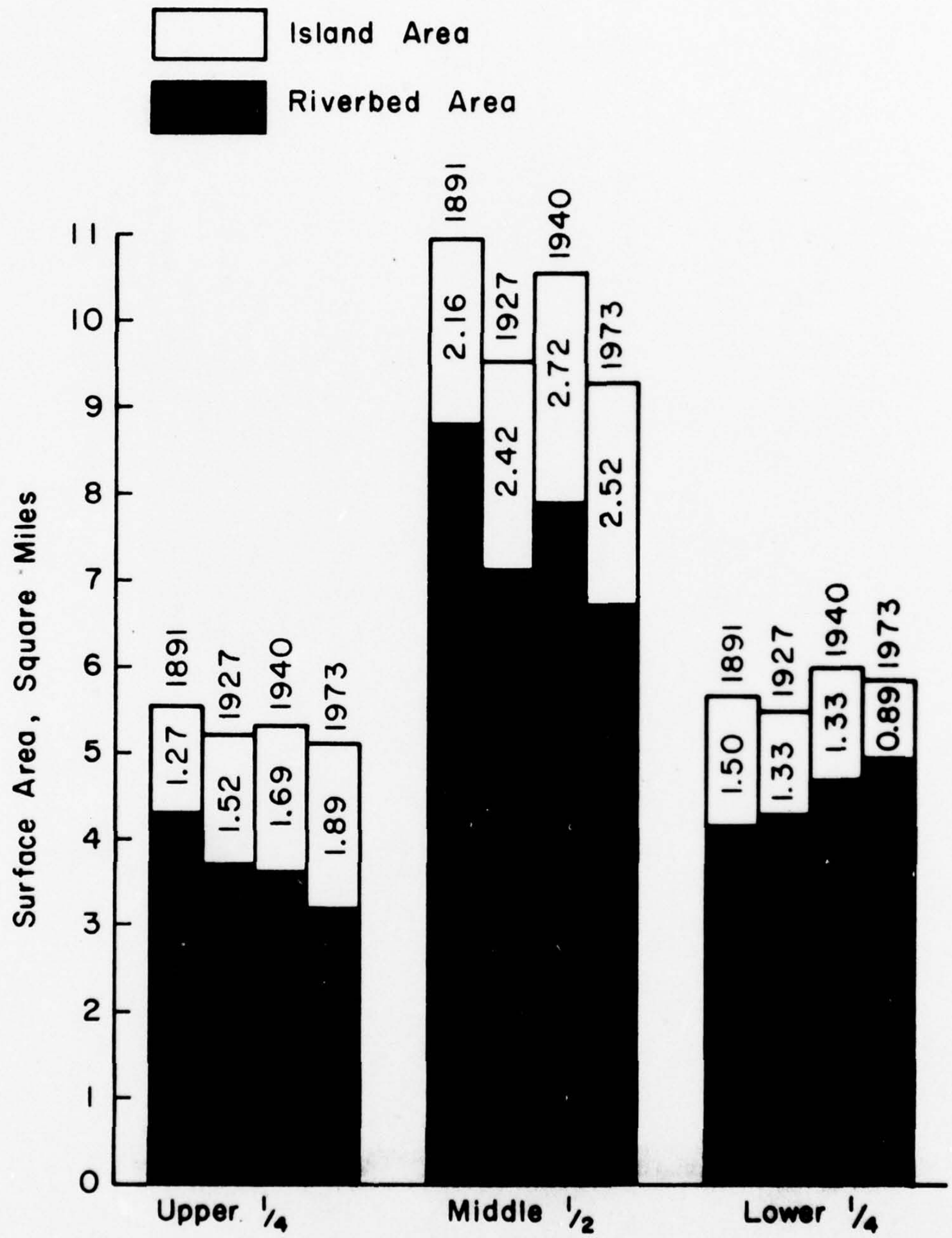


FIGURE 50. Pool 24 Surface Areas.

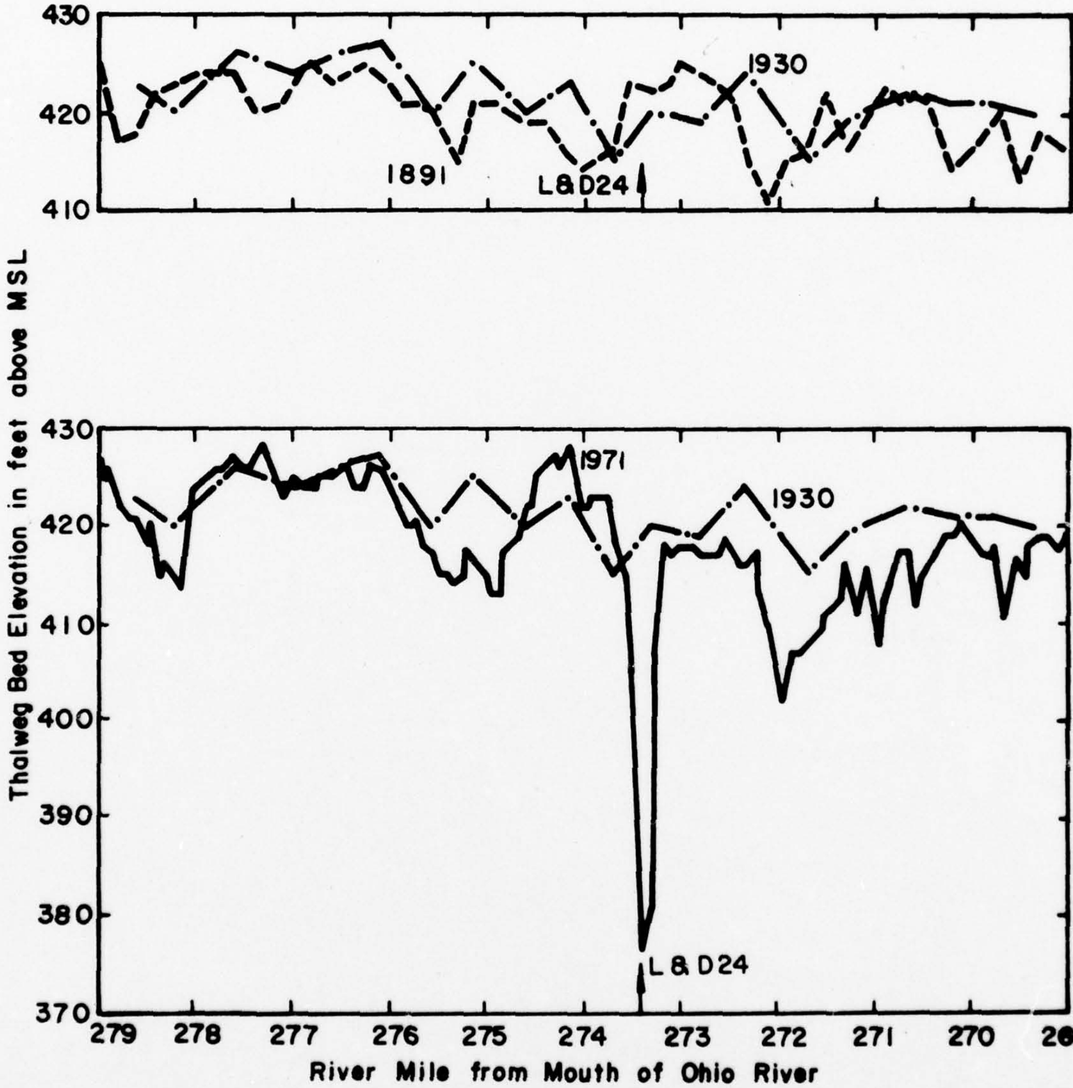


FIGURE 51. Longitudinal Profiles through Lock and Dam 24.

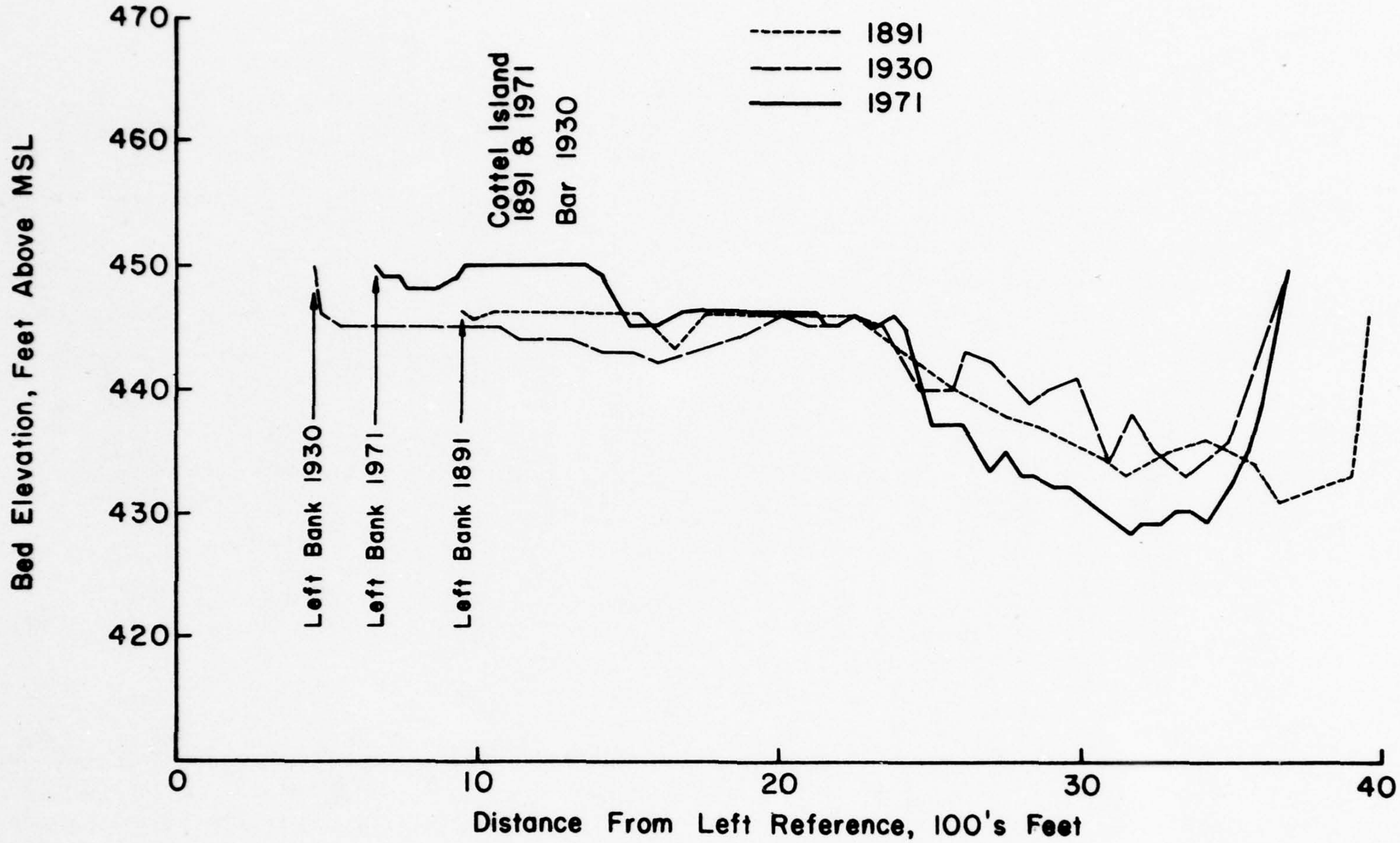


FIGURE 52. Cottel Island Cross Sections (RM 300).

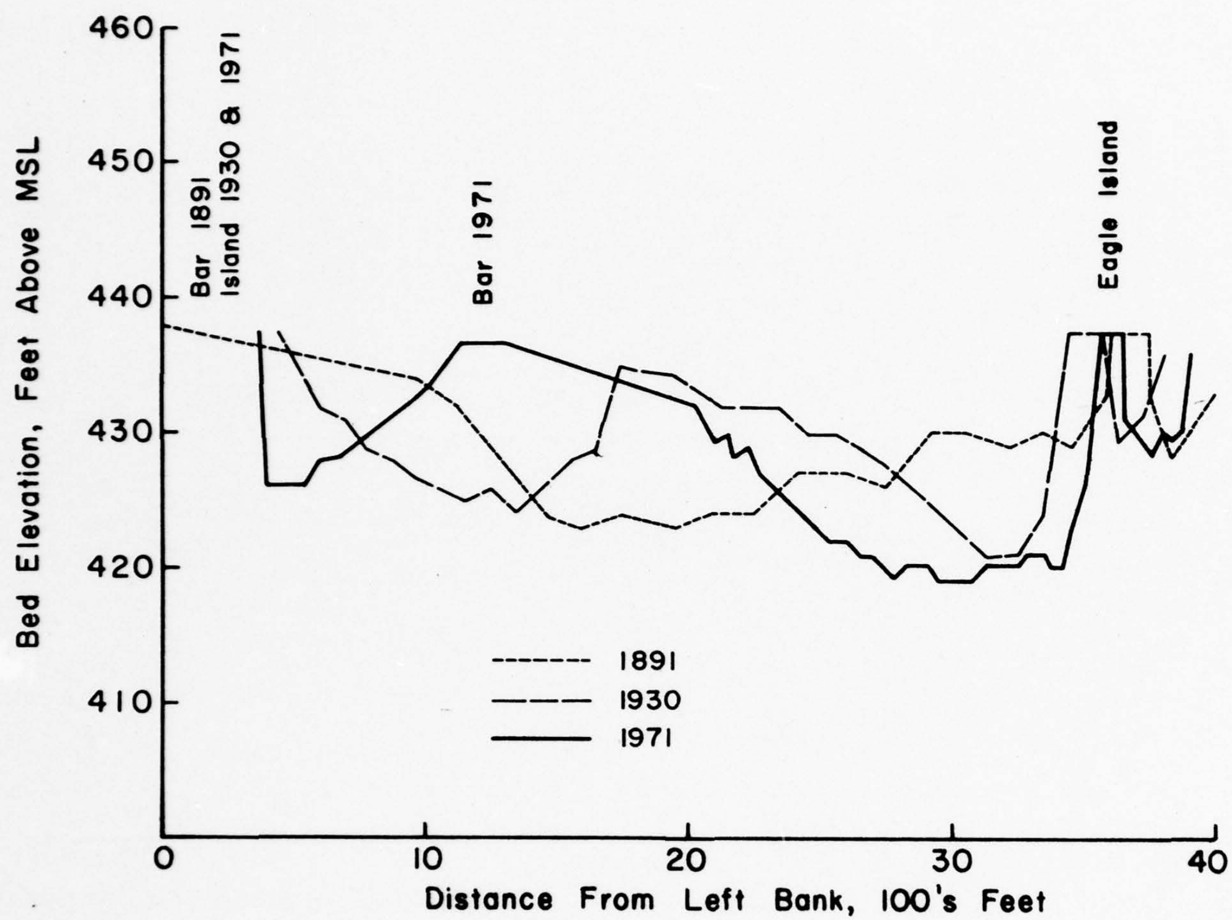


FIGURE 53. Eagle Island Cross Section (RM 270).

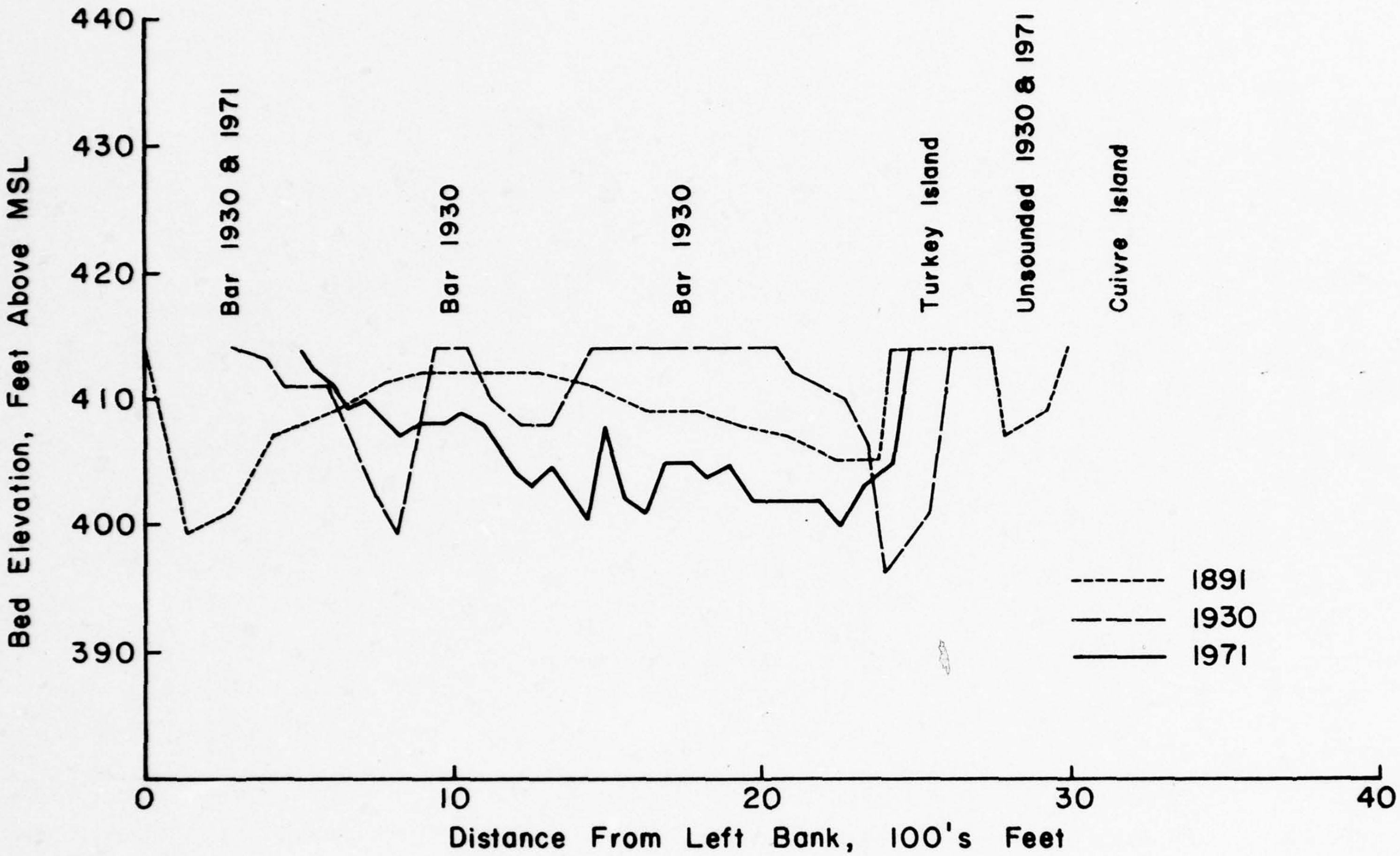


FIGURE 54. Turkey Island Cross Section (RM 236.9).

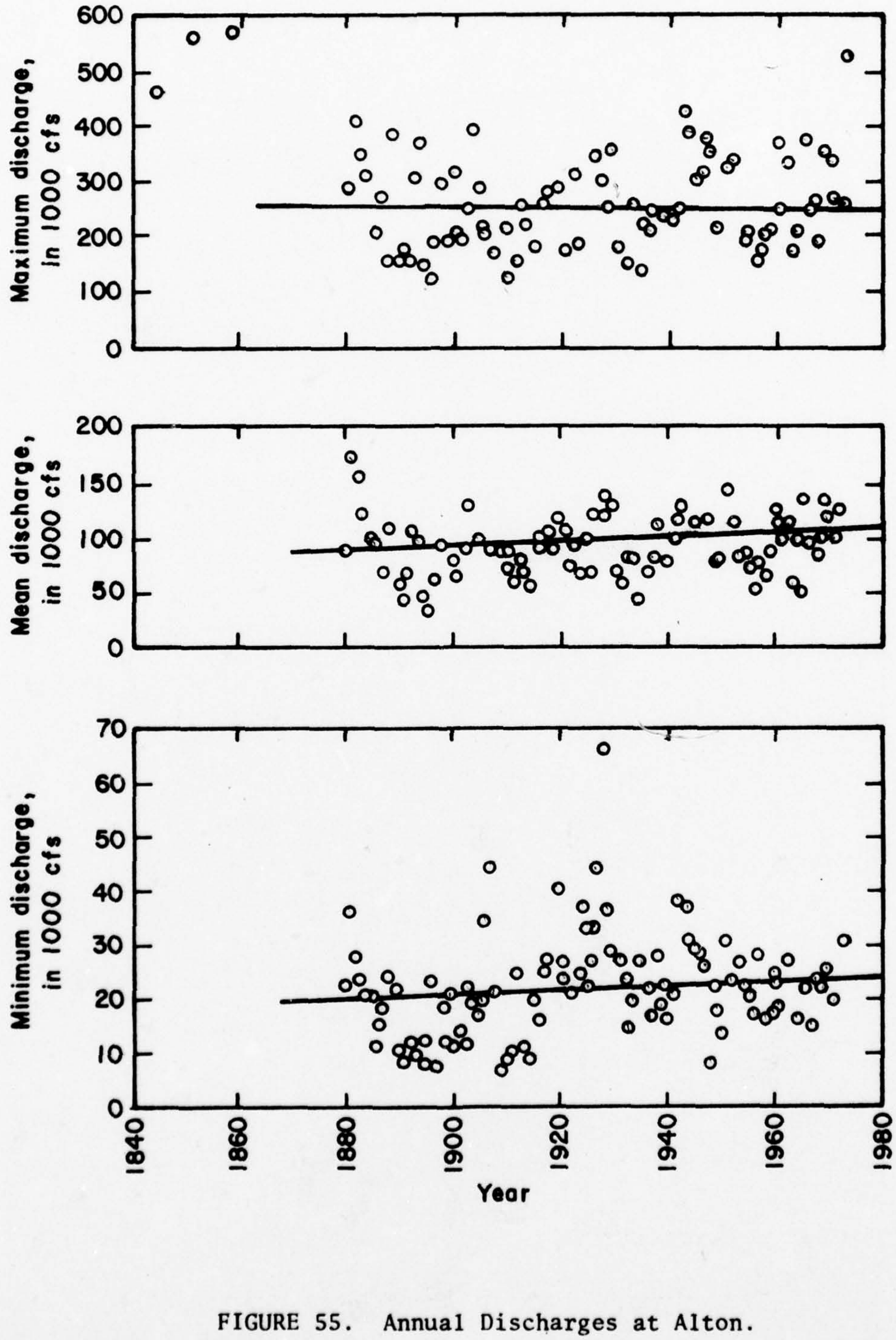


FIGURE 55. Annual Discharges at Alton.

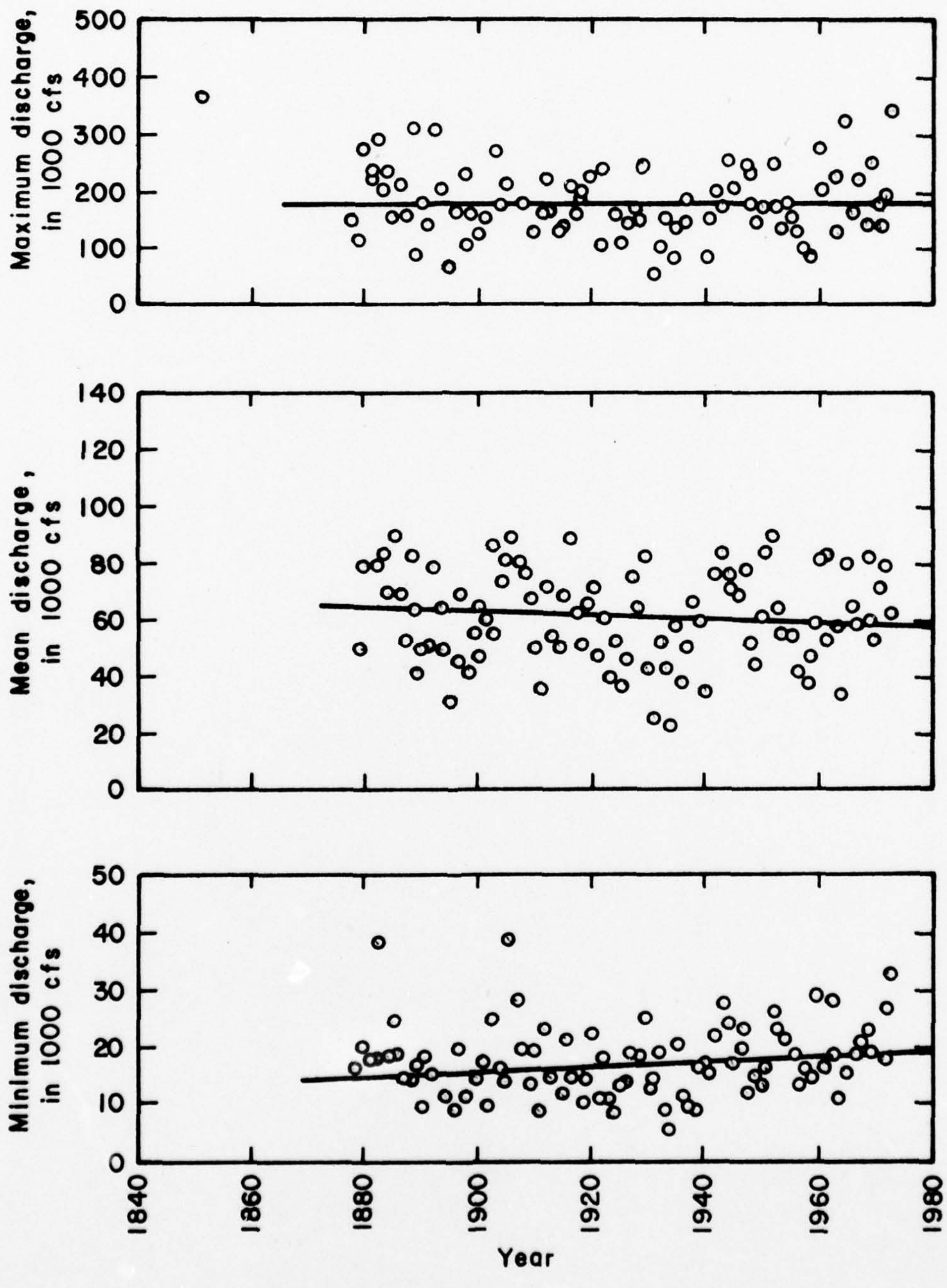


FIGURE 56. Annual Discharges at Keokuk.

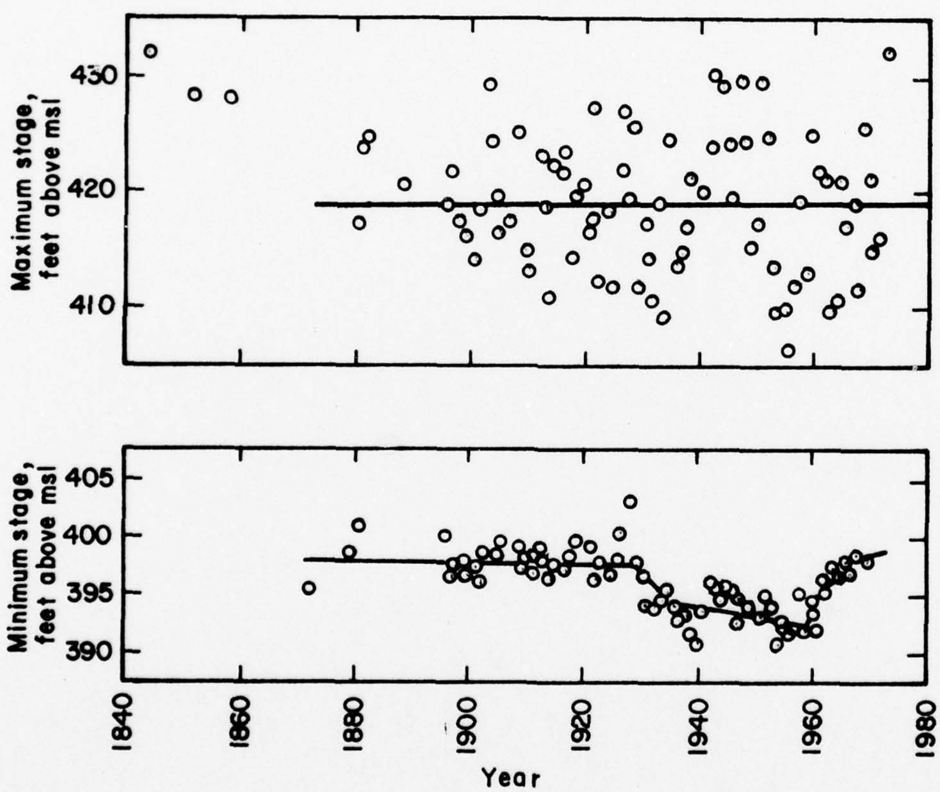


FIGURE 57. Annual Stages at Alton.

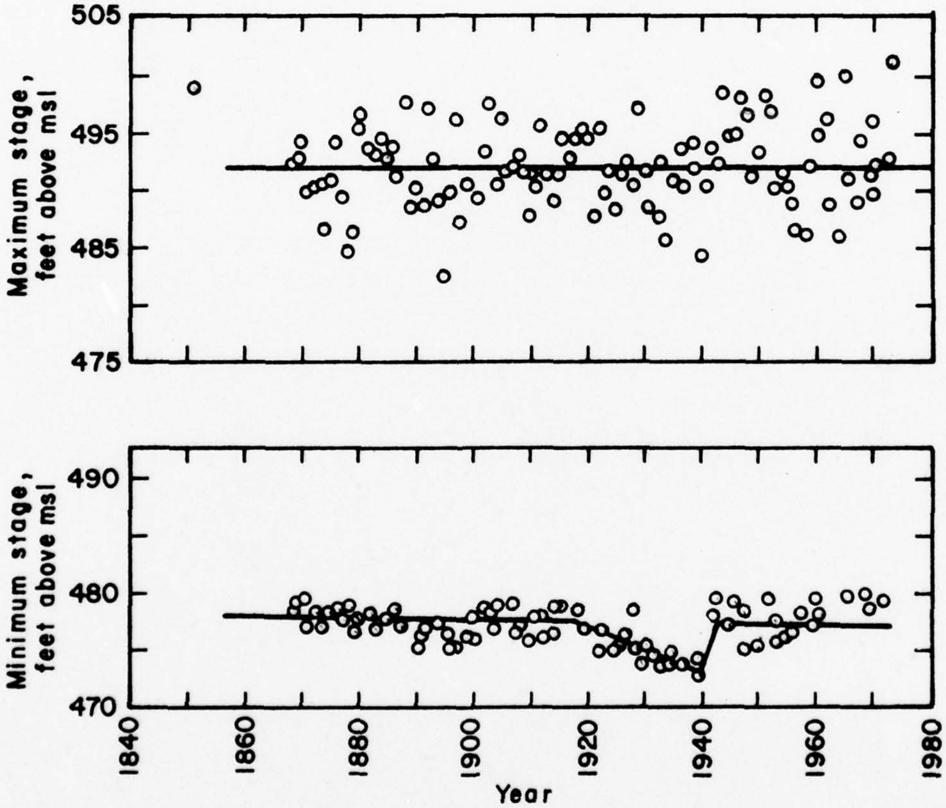
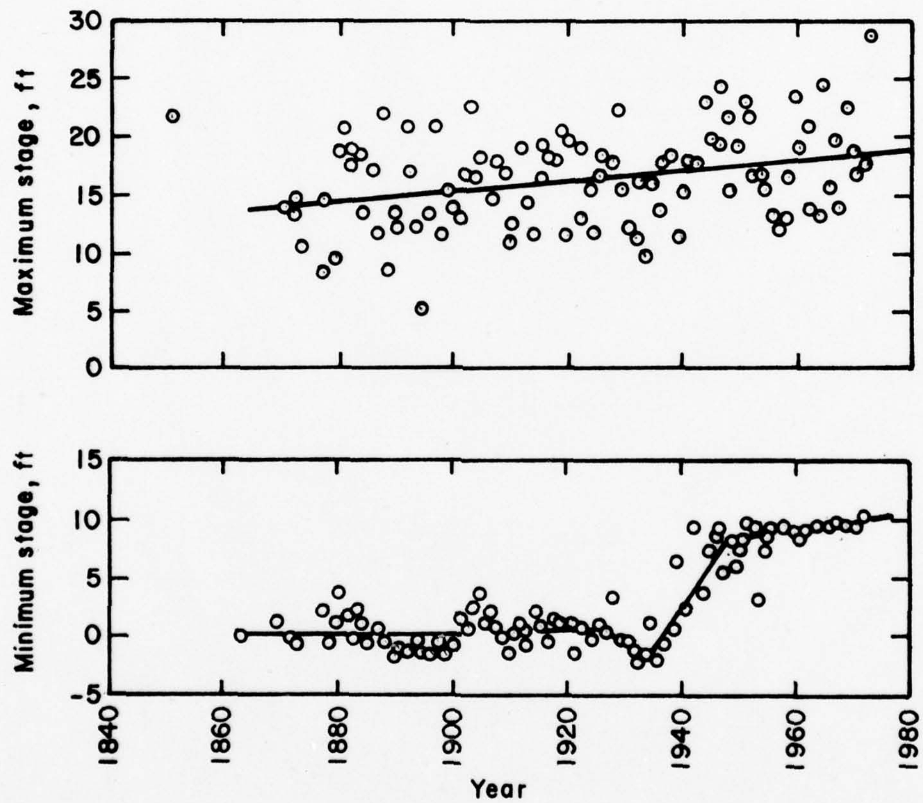
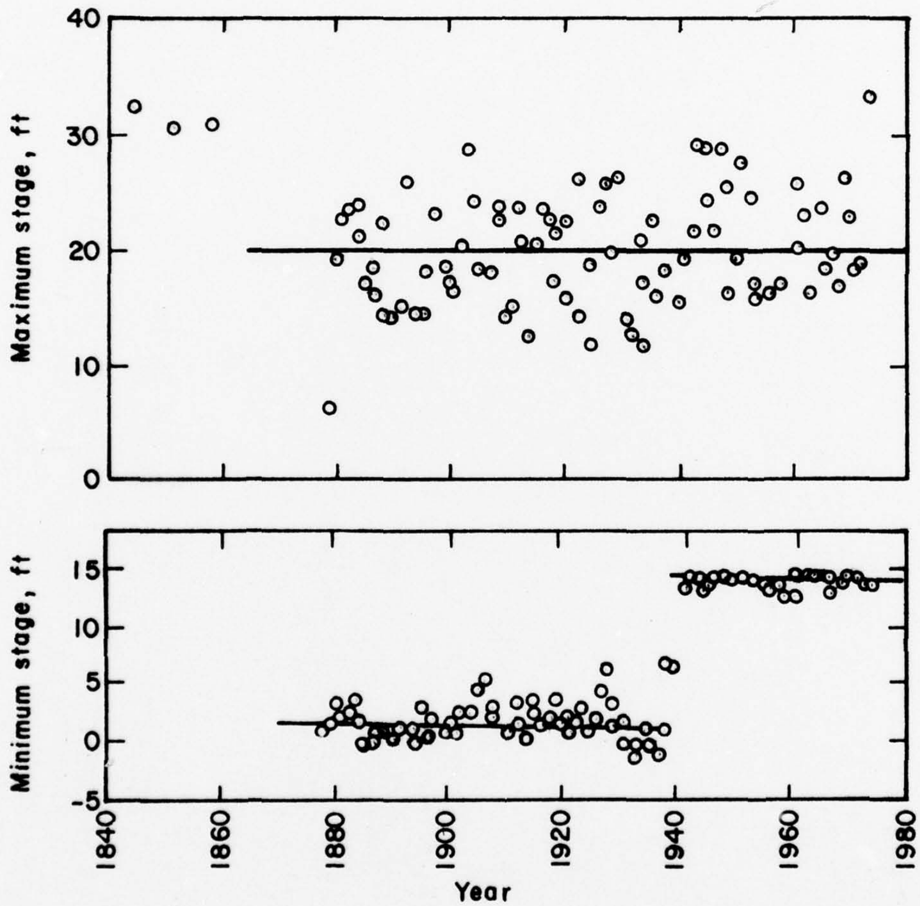


FIGURE 58. Annual Stages at Keokuk.



Note: Zero on the Hannibal gage is at 449.07 ft above msl.

FIGURE 59. Annual Stages at Hannibal.



Note: Zero on the Grafton gage is 403.79 ft above msl.

FIGURE 60. Annual Stages at Grafton.

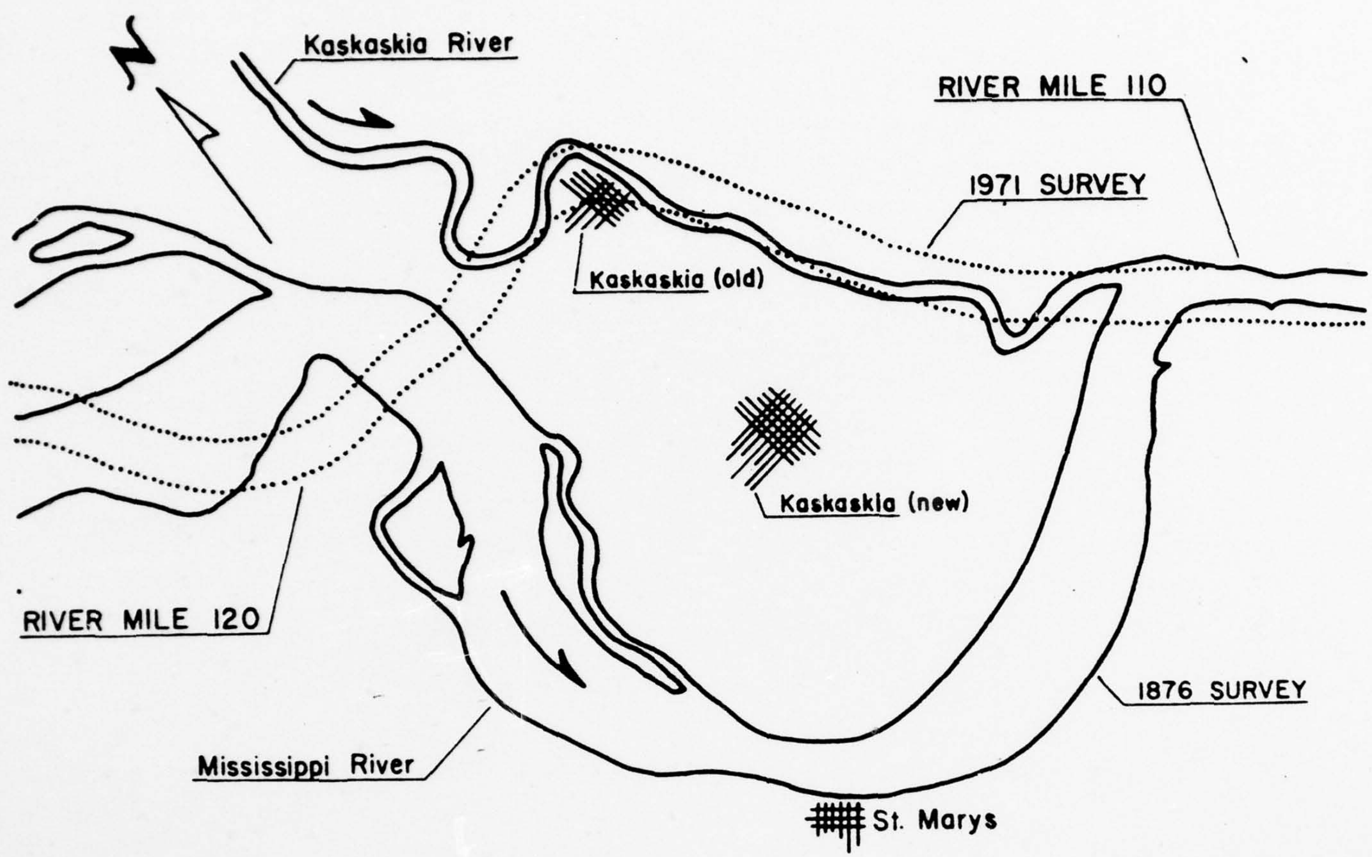


FIGURE 61. Capture of Kaskaskia River by Mississippi River in 1881 (after Degenhardt, 1973).

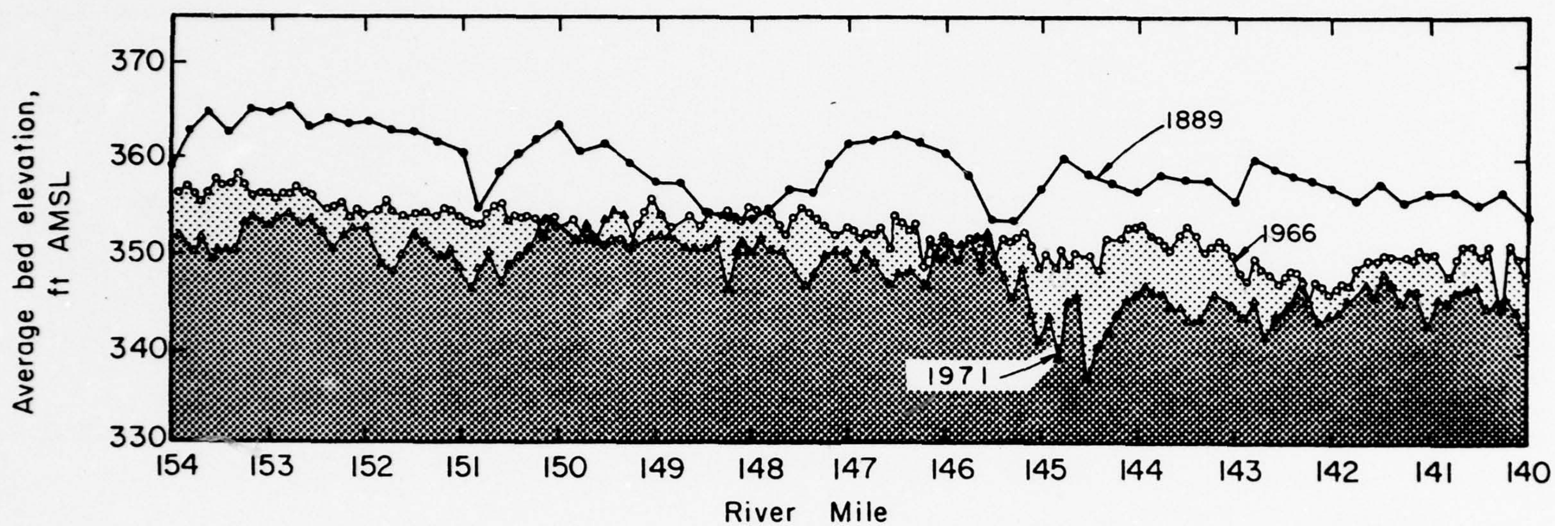


FIGURE 62. Bed Elevations Miles 140 to 154 (after Simons et al., 1974).

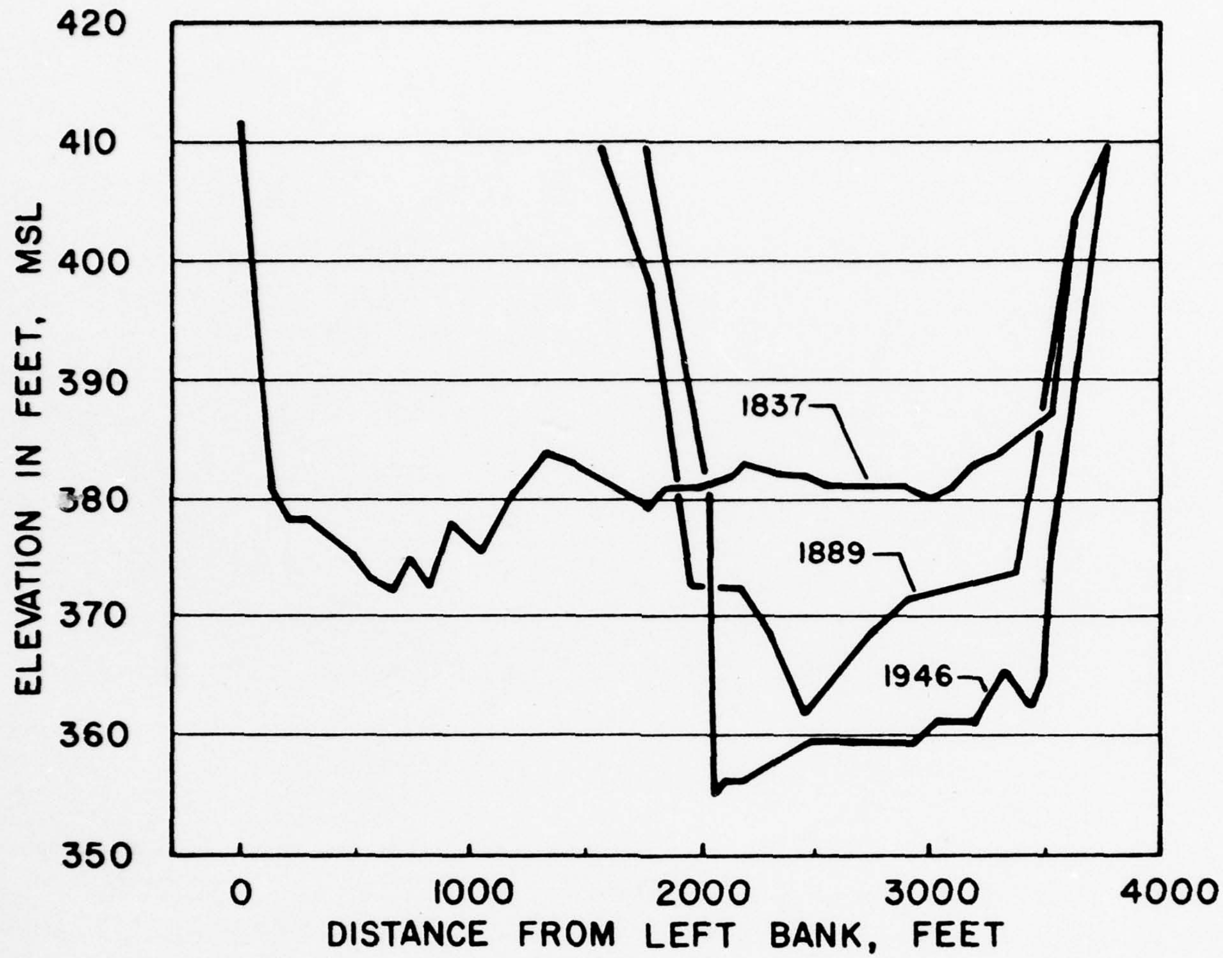


FIGURE 63. Comparative Channel Cross Sections at St. Louis, Looking Downstream, Mile 180 (after Degenhardt, 1973).

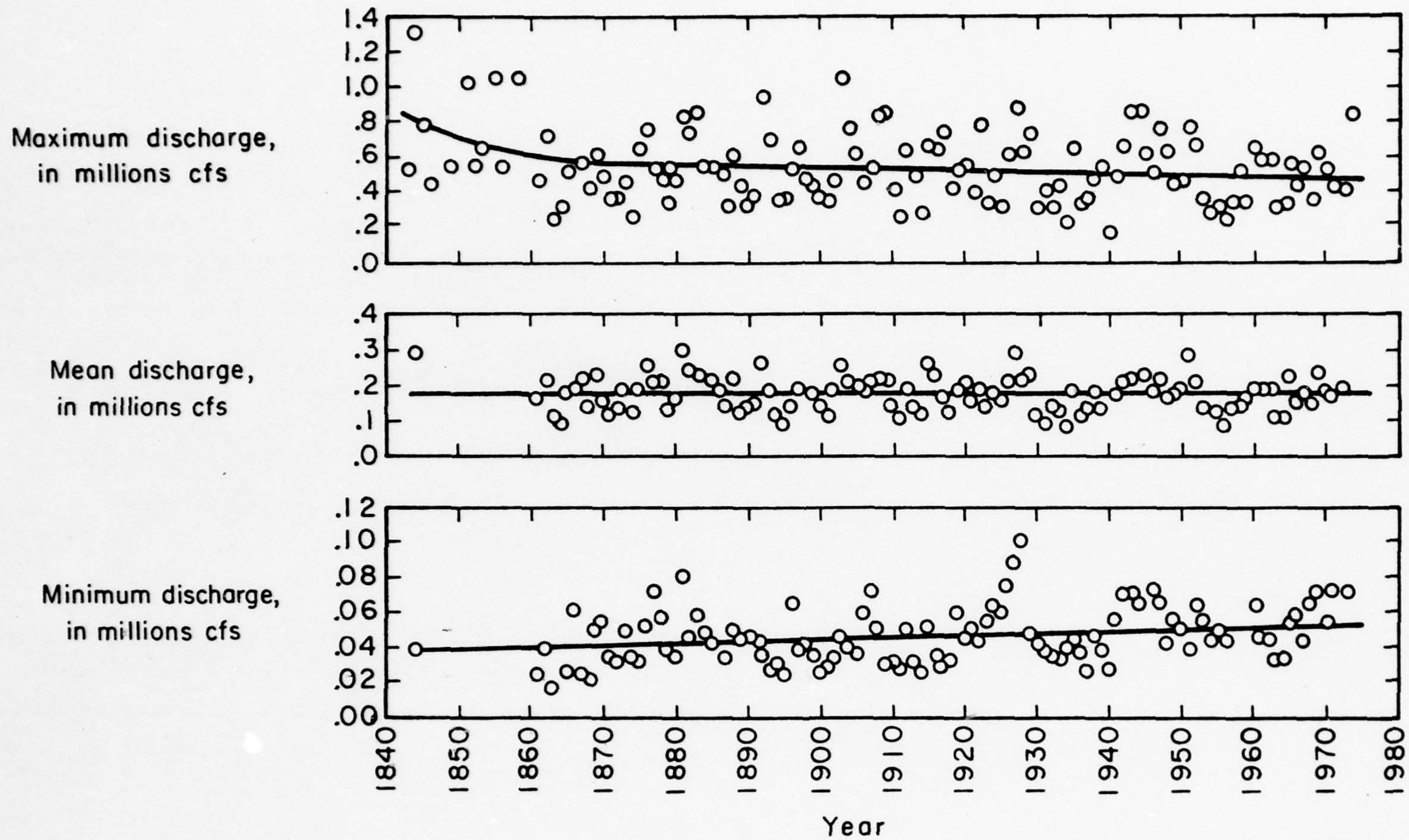


FIGURE 64. Discharges at St. Louis (after Simons et al., 1974).

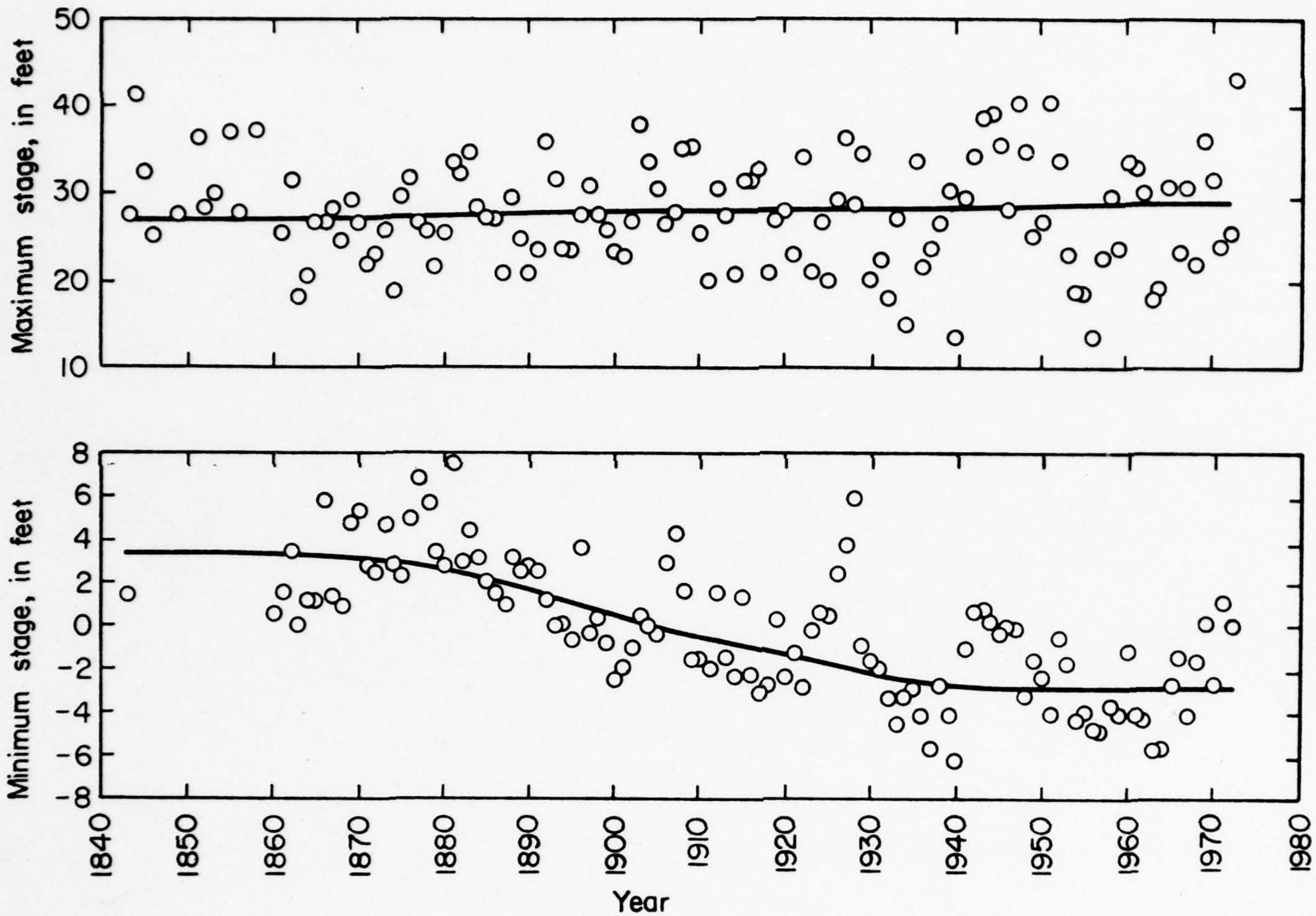


FIGURE 65. Stages at St. Louis (after Simons et al., 1974).

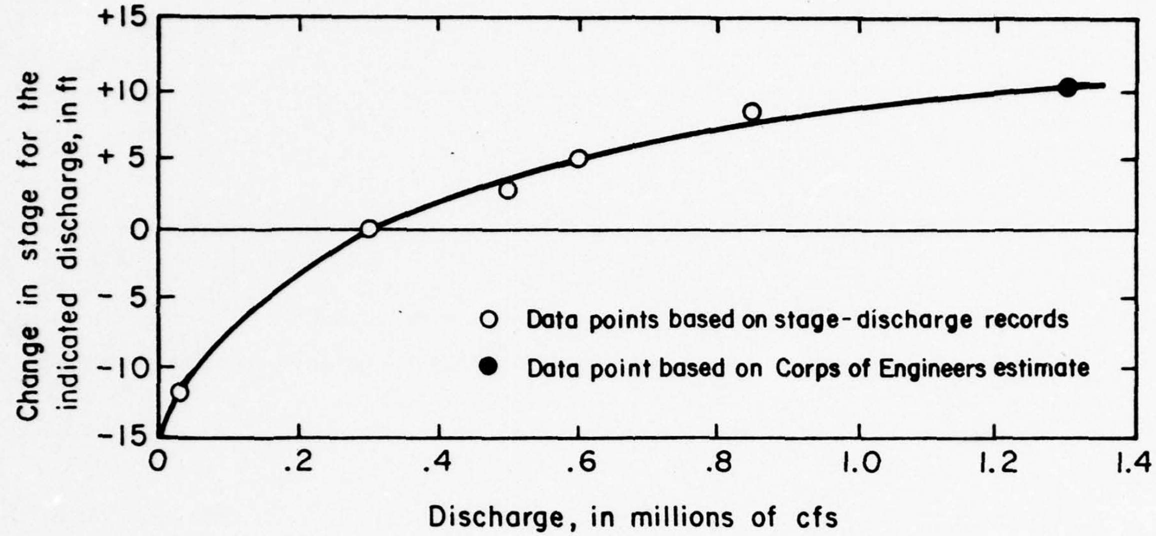
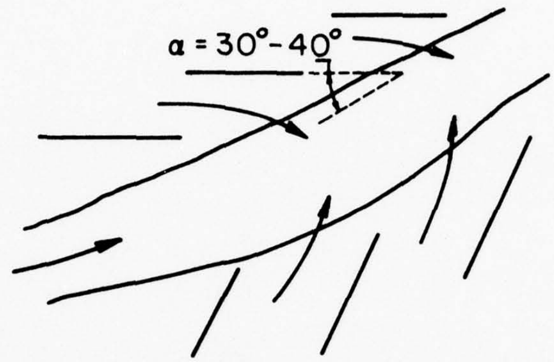
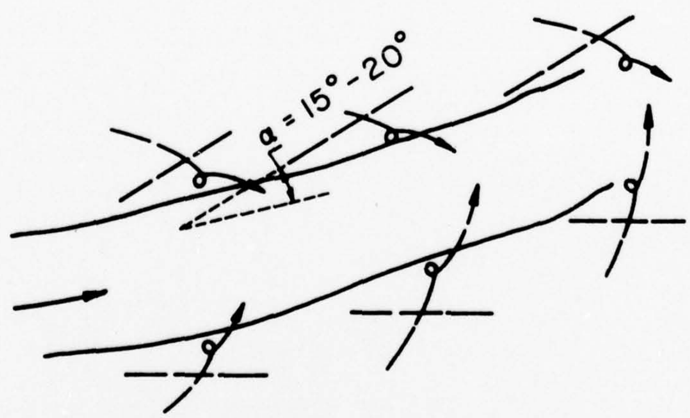


FIGURE 66. Changing Stages at St. Louis (after Simons et al., 1974).

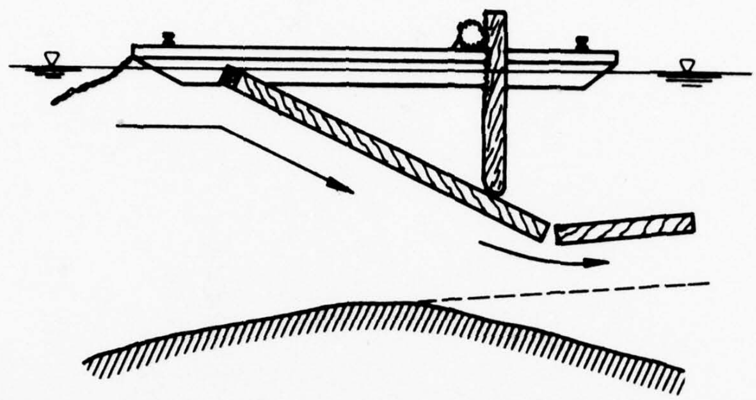


a) Orientation of Bandals

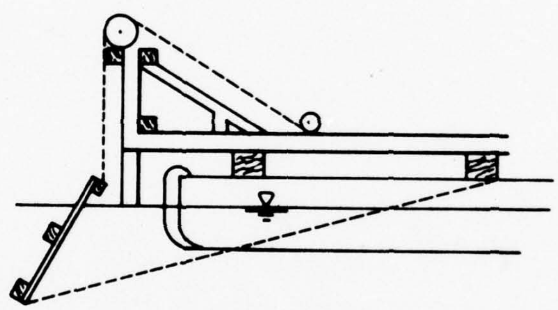


b) Orientation of Bottom Panels

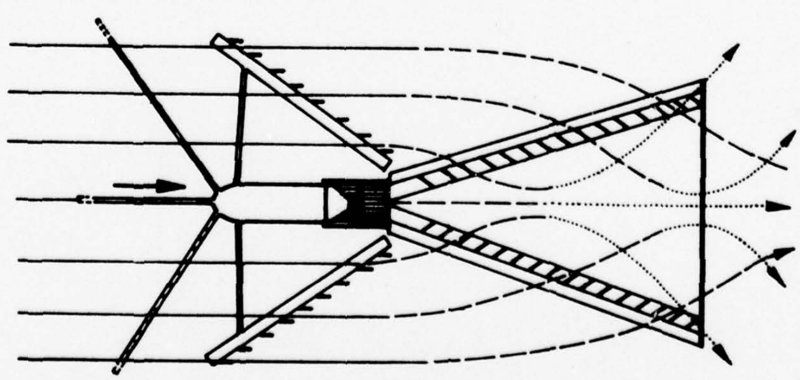
FIGURE 67. Temporary River Works (after NEDECO, 1959).



a) The "Machine for Deepening River Channels" (after Ockerson, 1898)



b) The "Kingston" Deflector (after Ockerson, 1898)



c) The Prostov Dredging Barge (after NEDECO, 1959)

FIGURE 68. Current-Deflector Dredges.

LEGEND

1 Waterline
2 Bottom
3 Hull
4 Deckhouse
5 Lever Room
6 Ladder with Suction Pipe
7 Cutter Head
8 Dredge Pump
9 Spud
10 Spud Frame
11 Discharge Pipeline
12 Pontoon

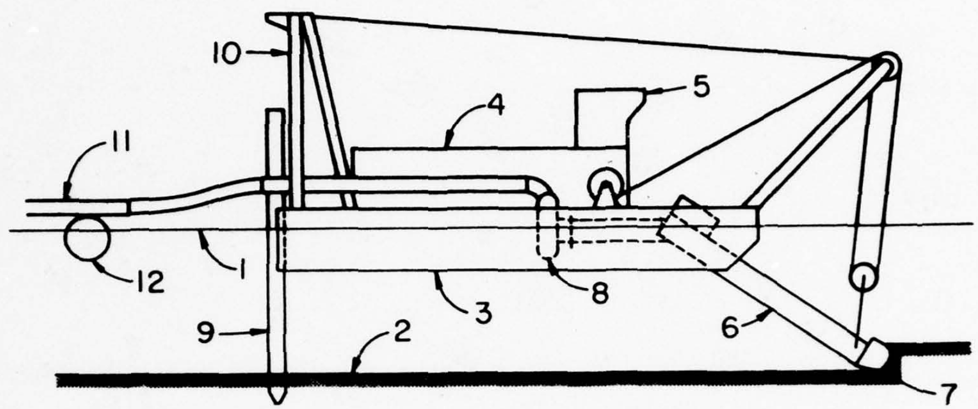


FIGURE 69. Cutterhead Dredge Components (after Black, 1973).

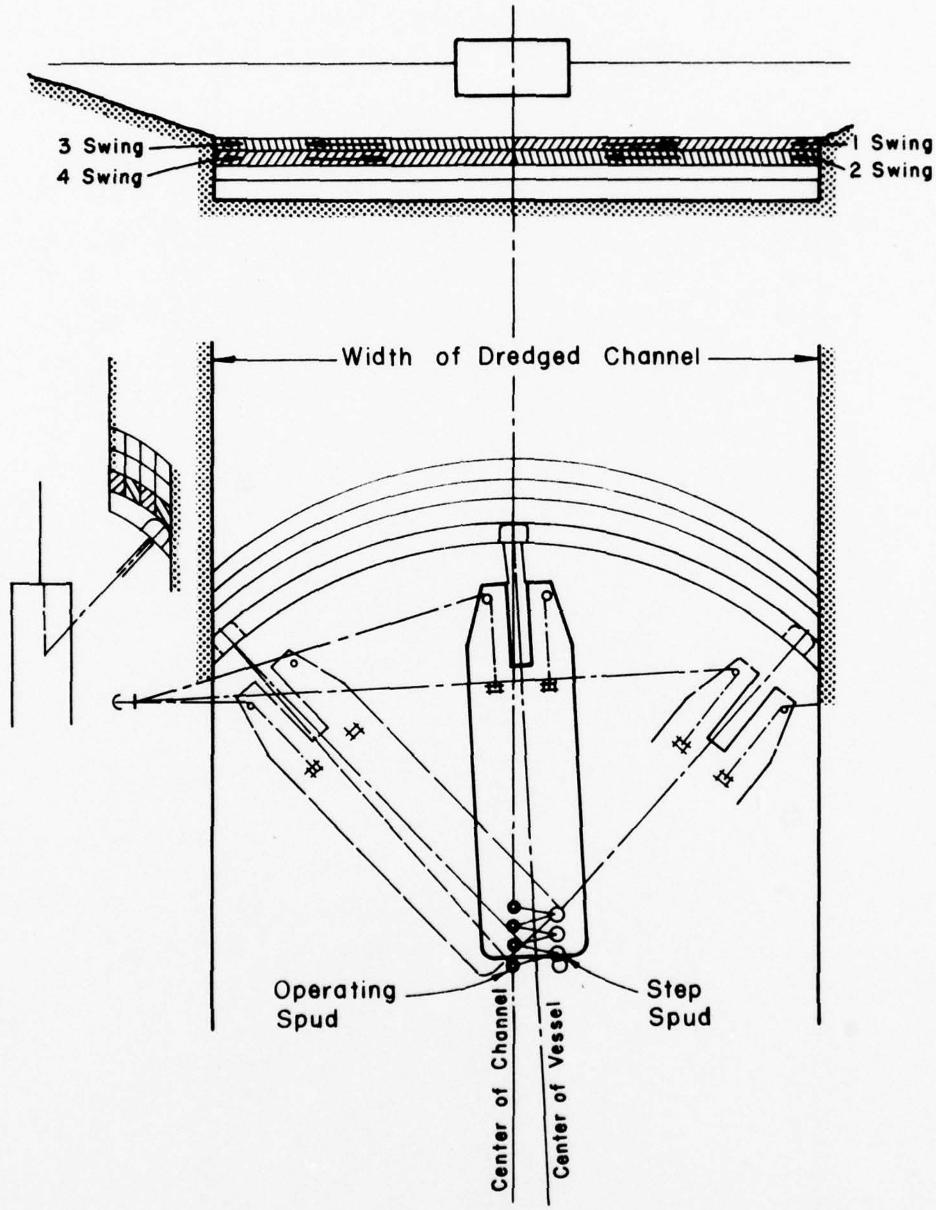


FIGURE 70. Operation of a Cutterhead Dredge (after NEDECO, 1965).

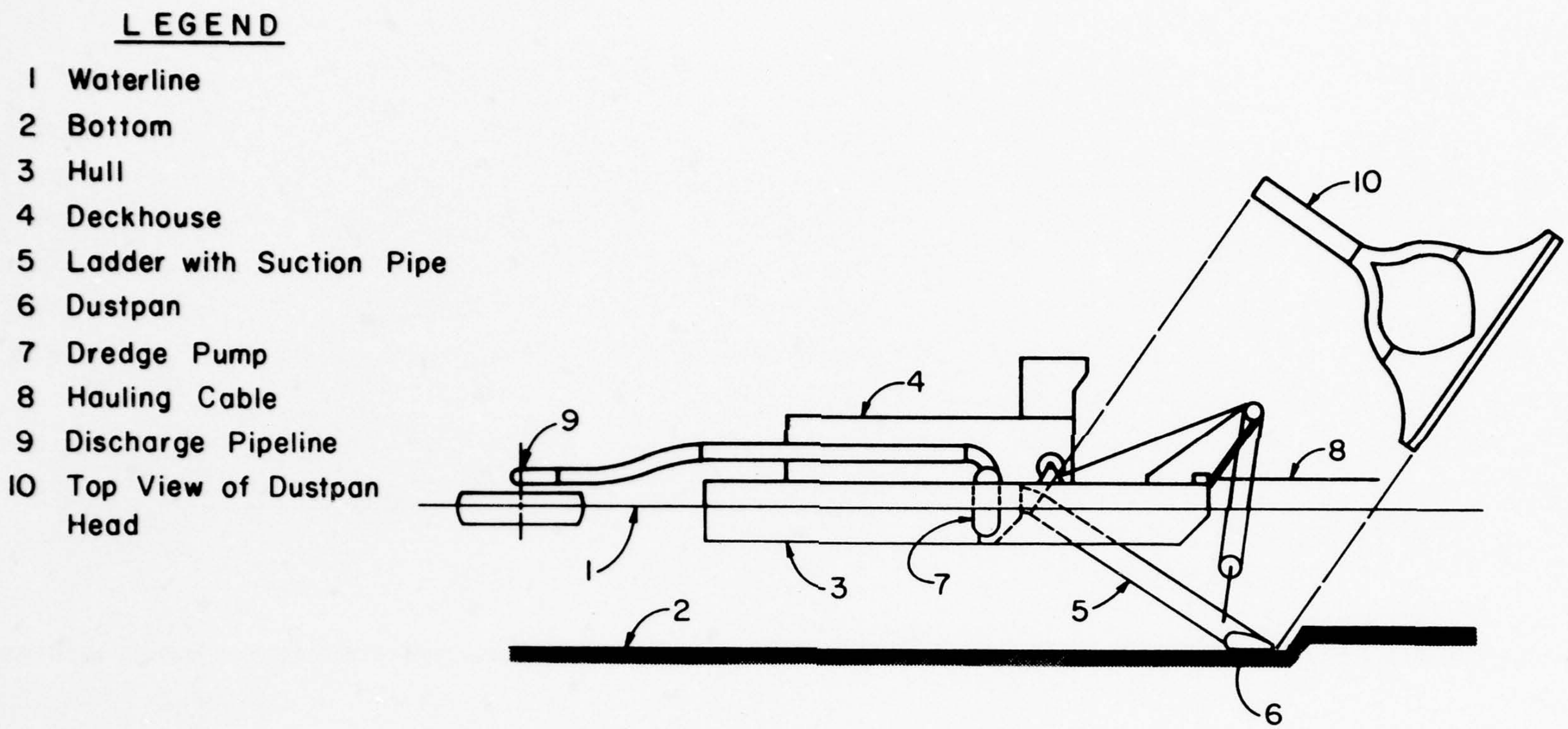


FIGURE 71. Dustpan Dredge Components (after Black, 1973).

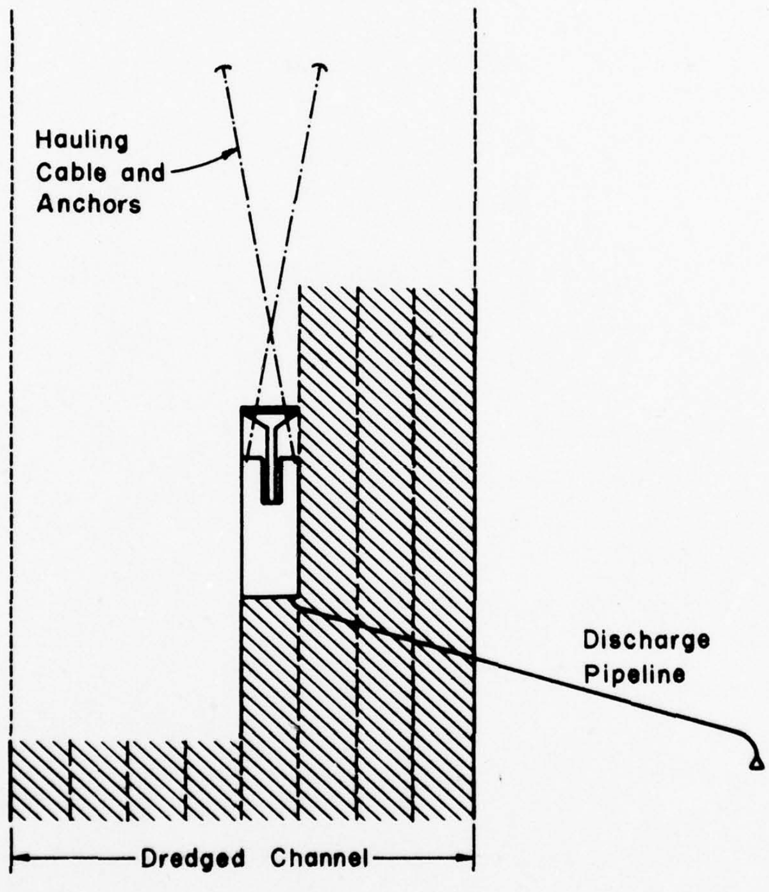


FIGURE 72. Operation of a Dustpan Dredge (after NEDECO, 1965).

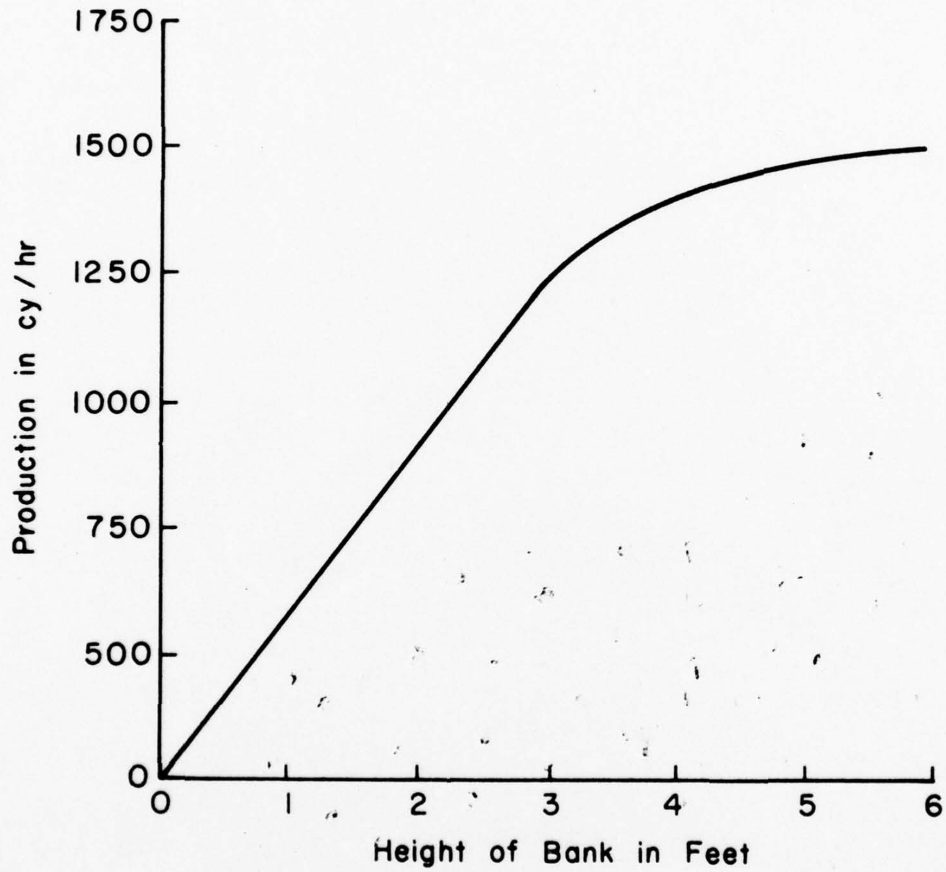


FIGURE 73. Production Curve for 30-inch Hydraulic Cutterhead Dredge Pumping through 2000-3000 Feet of Discharge Pipeline (after Hyde and Beeman, 1963).

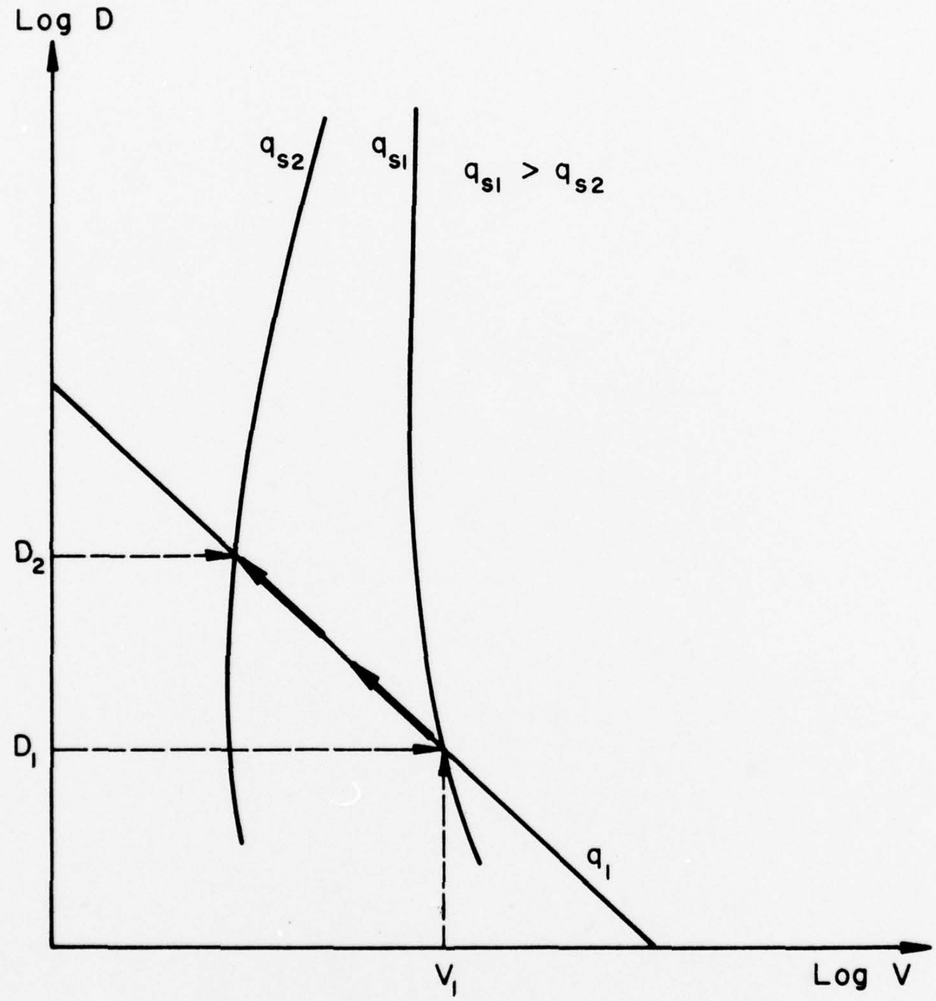


FIGURE 74. Colby Transport Relations Replotted for Width and Depth Change Analysis (after Nordin, 1971).

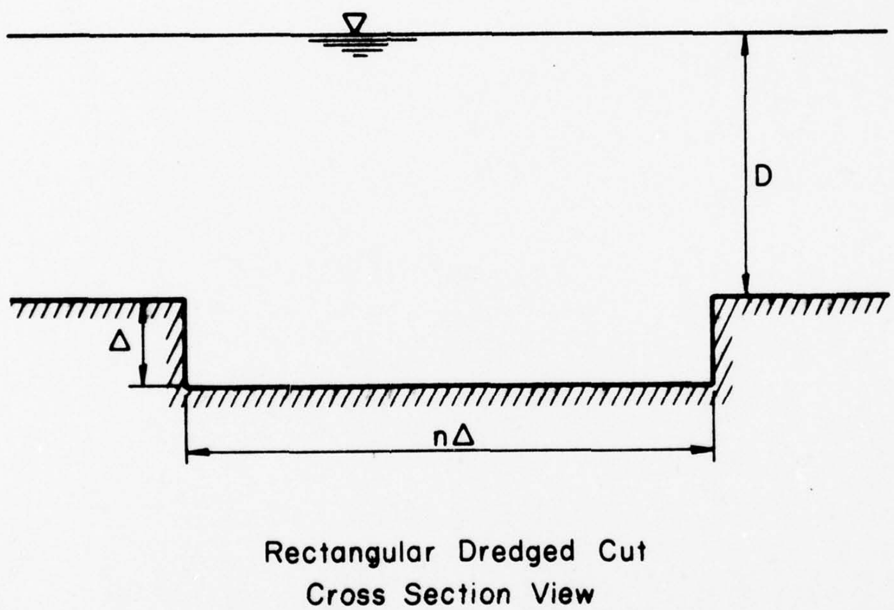


FIGURE 75. Definition Sketch--Rectangular Dredged Cut.

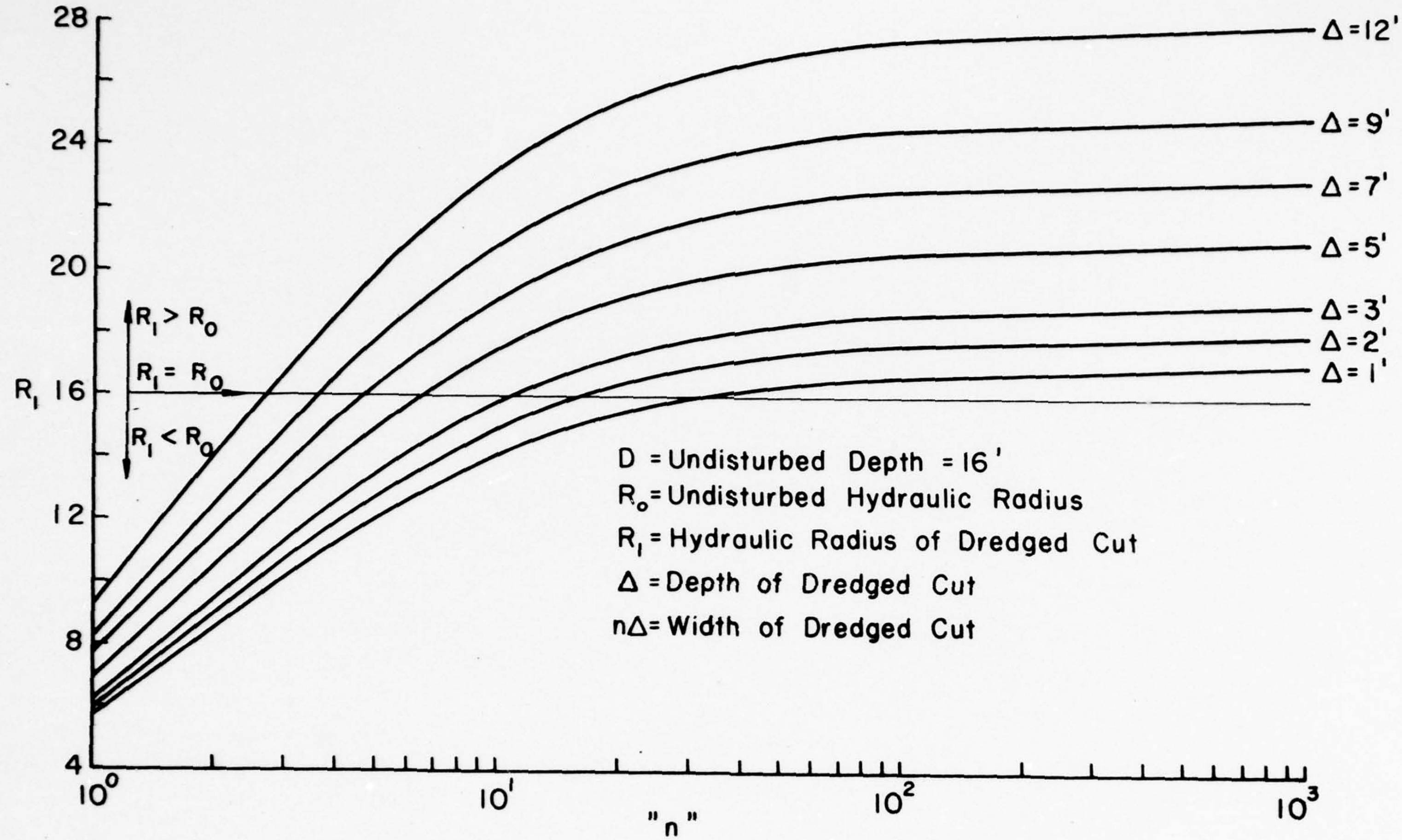


FIGURE 76. Rectangular Dredged Cut Design Curve--Steady, Uniform Flow.

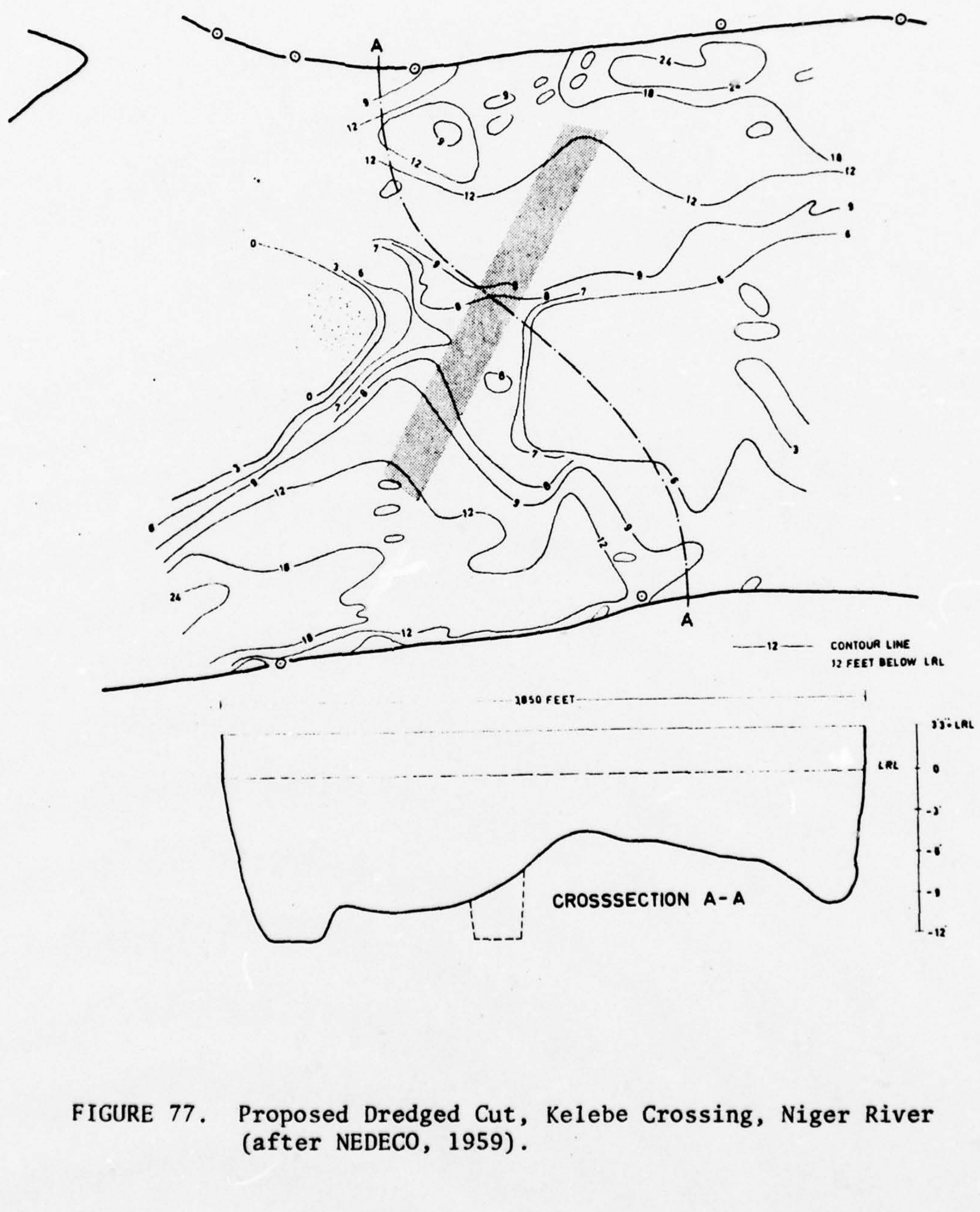


FIGURE 77. Proposed Dredged Cut, Kelebe Crossing, Niger River (after NEDECO, 1959).

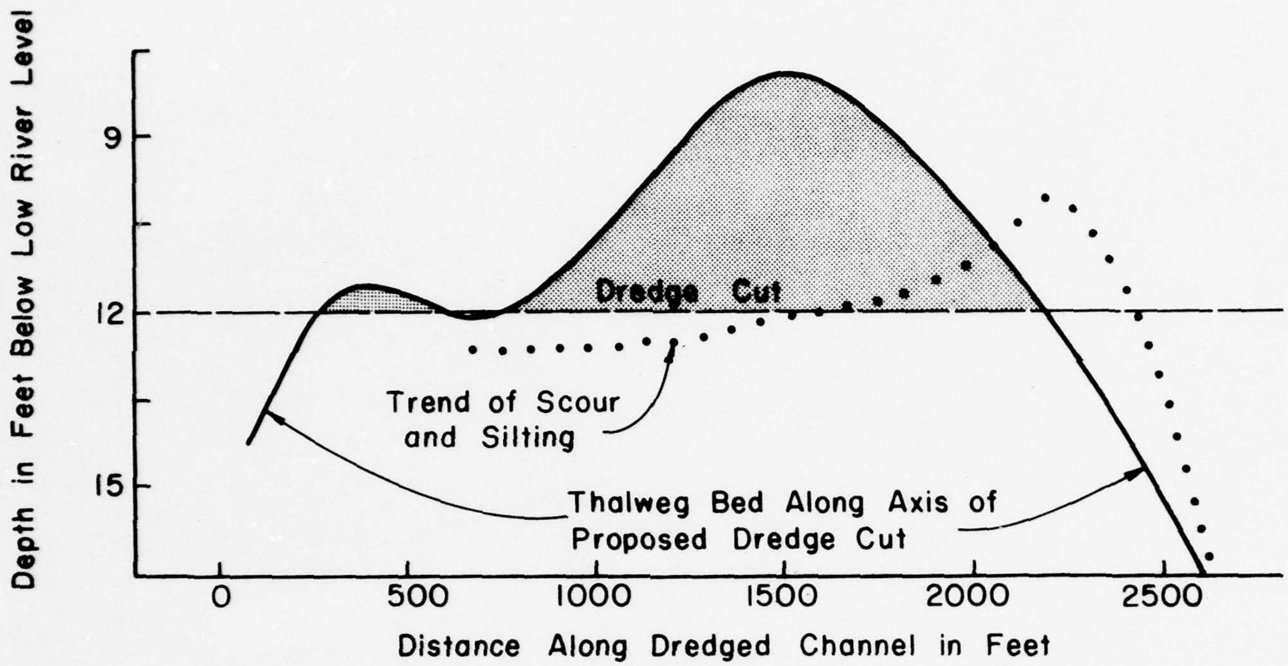


FIGURE 78. Scour and Fill in a Dredged Channel (after NEDECO, 1959).

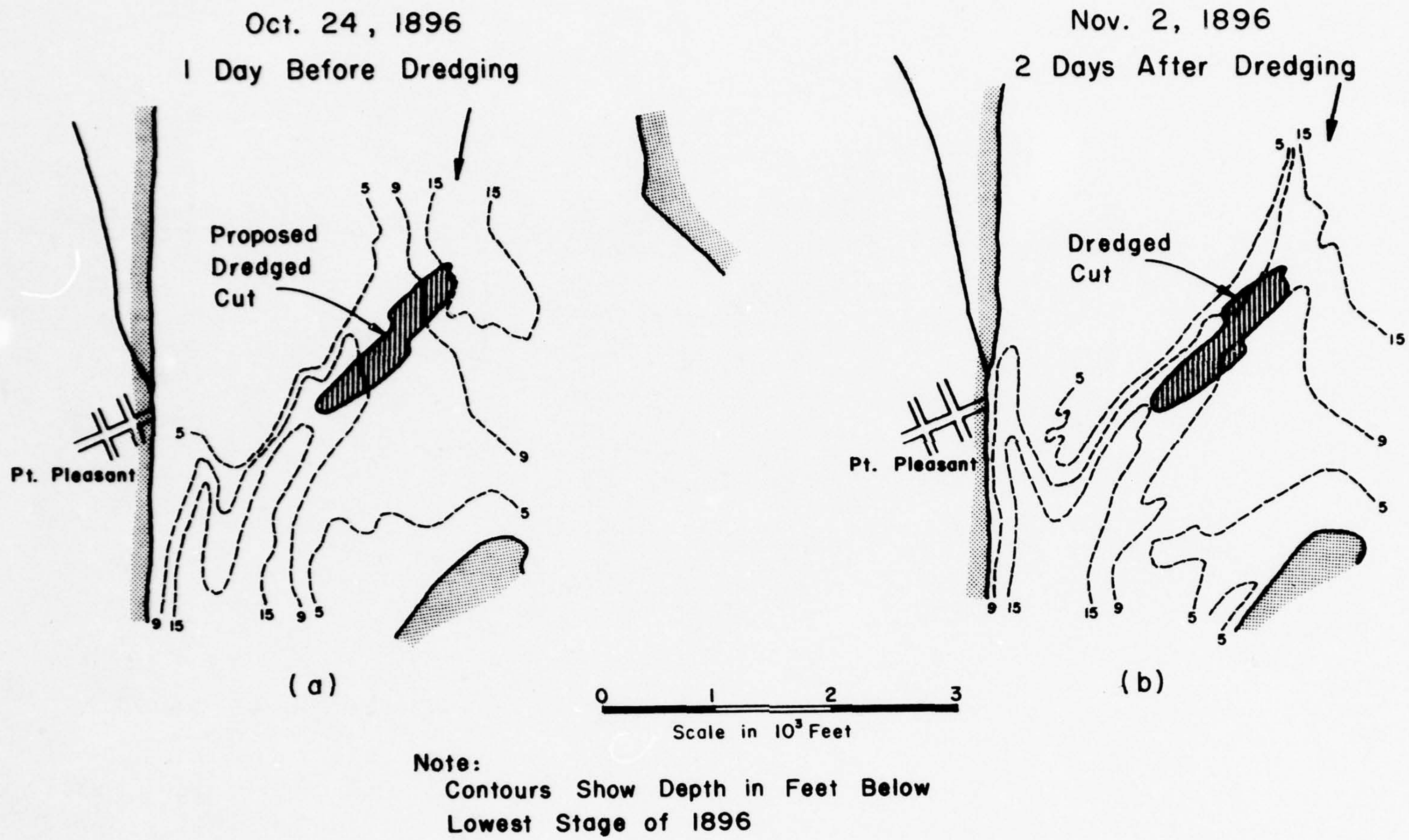


FIGURE 79. Dredging of the Lower Point Pleasant Bar, Lower Mississippi River (after Ockerson, 1898).

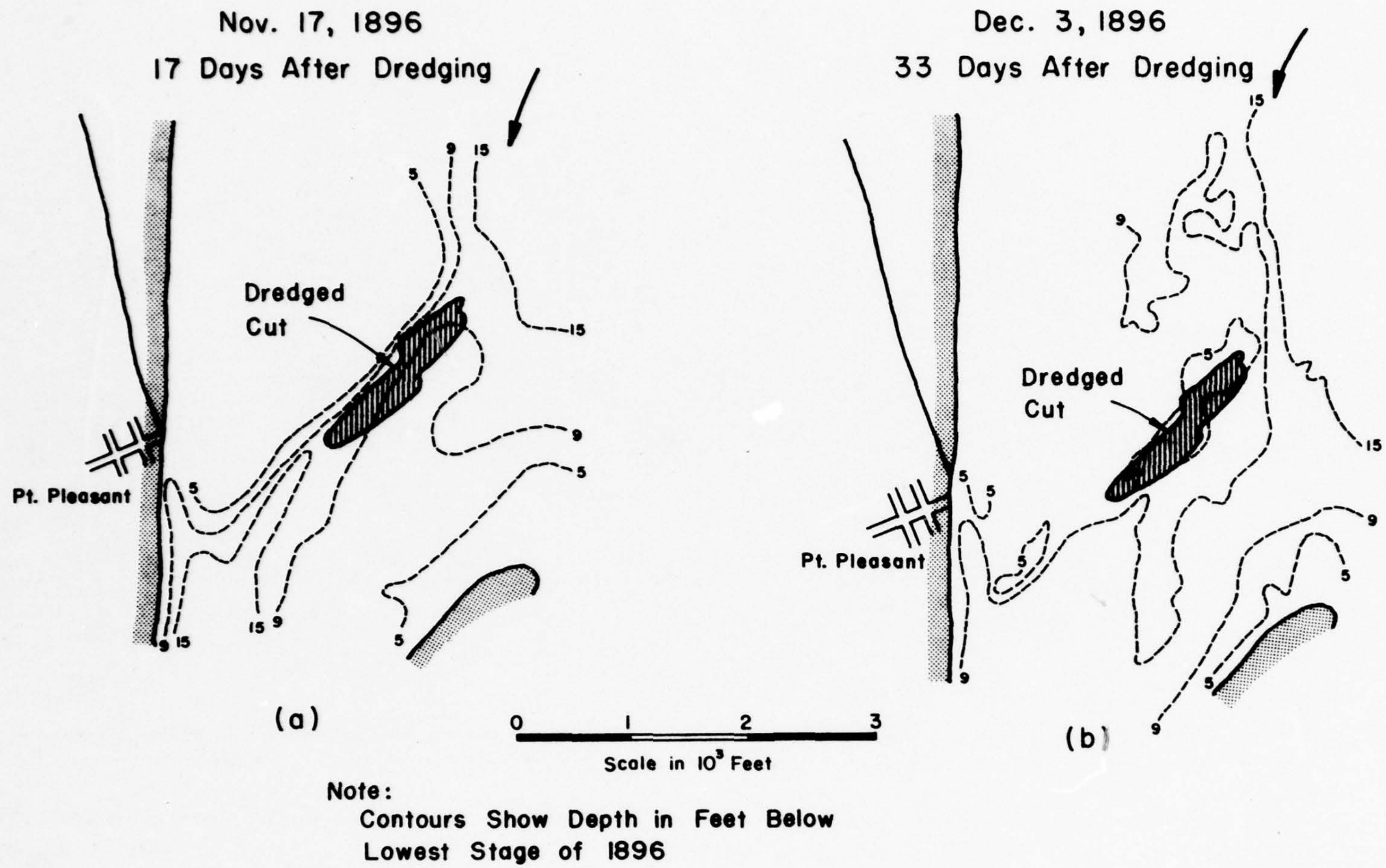


FIGURE 80. Dredging of the Lower Point Pleasant Bar, Lower Mississippi River (after Ockerson, 1898).

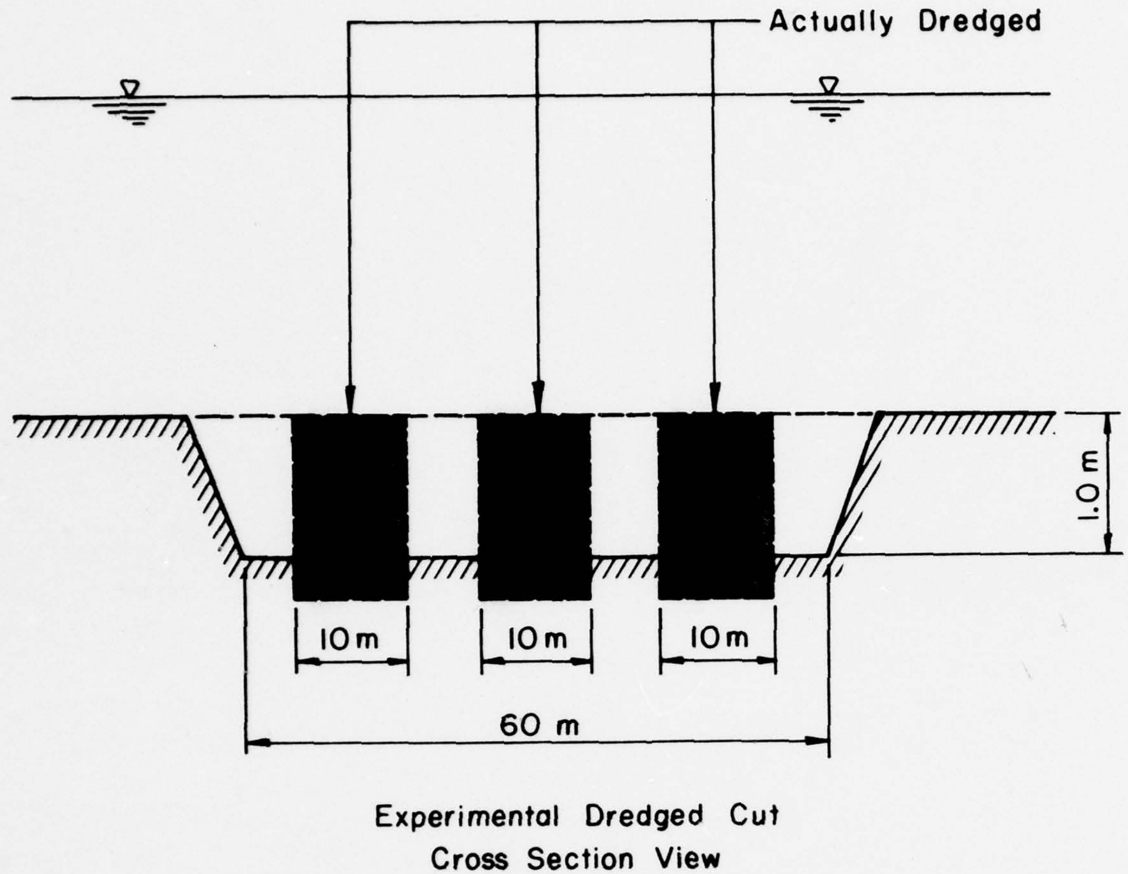


FIGURE 81. Experimental Dredged Cut, Rio Magdalena, Colombia (after NEDECO, 1973).

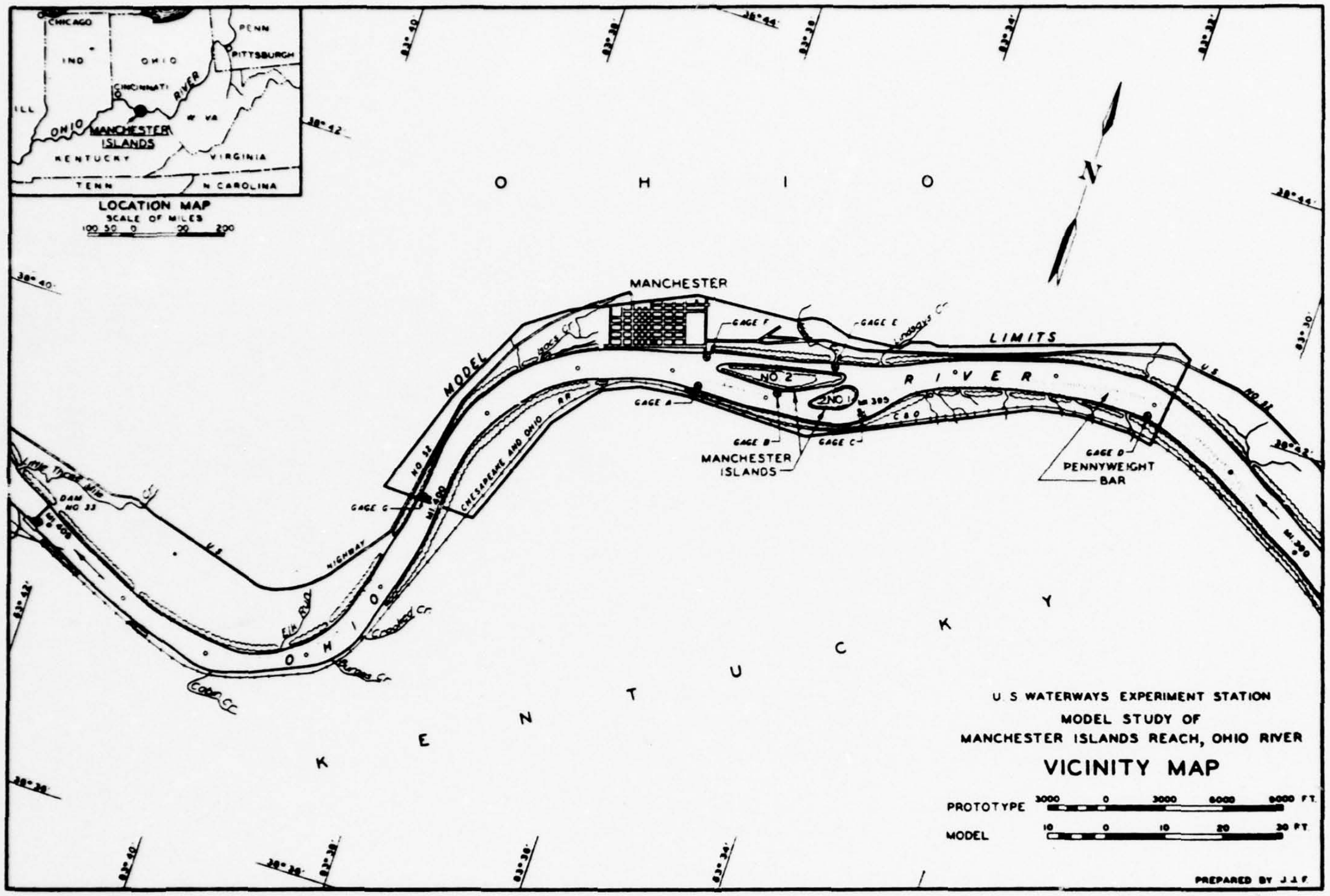


FIGURE 82. Manchester Island Reach and Model Limits (after Corps of Engineers, Waterways Experiment Station, 1941).

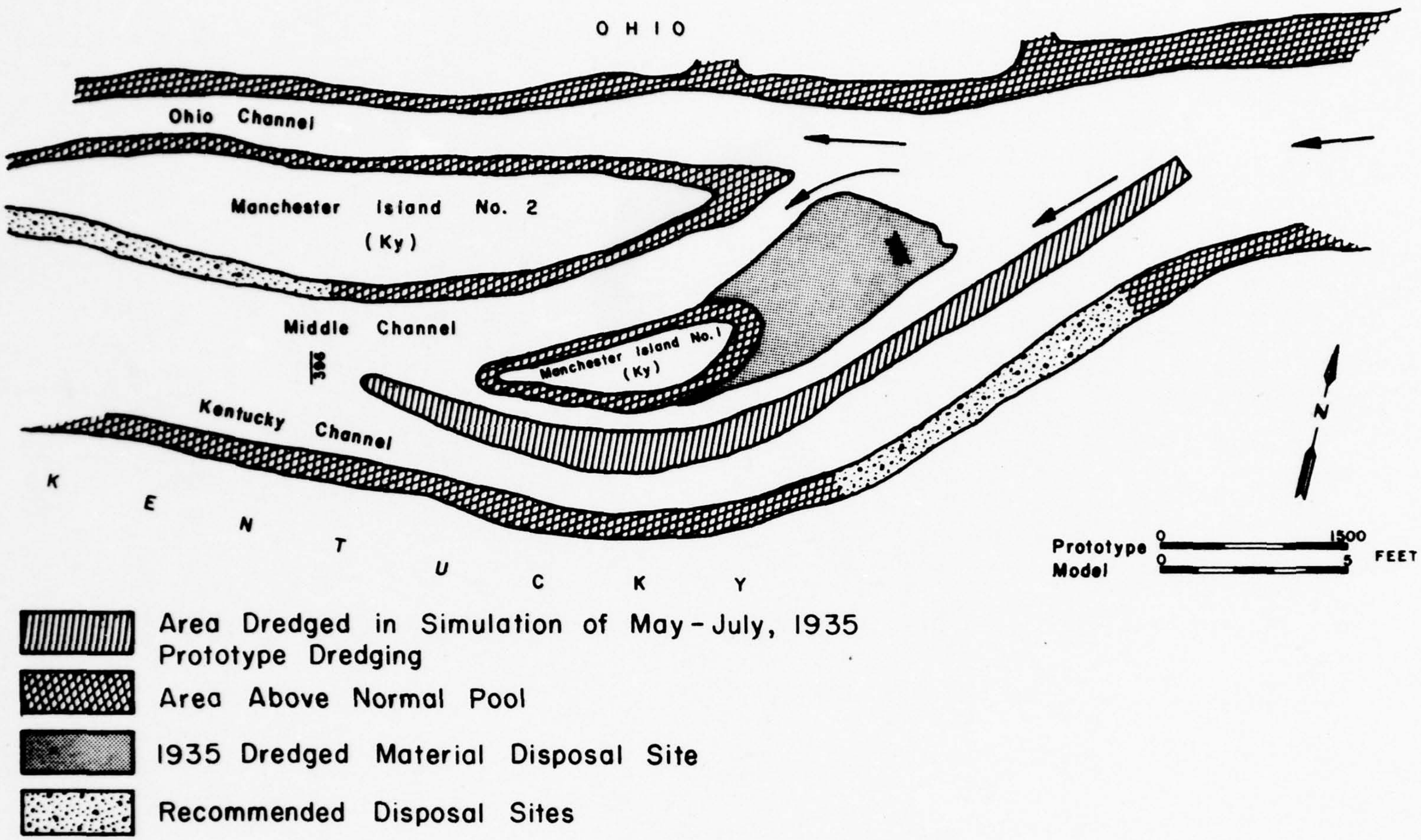


FIGURE 83. Existing Conditions--Manchester Islands Model Study (after Corps of Engineers, Waterways Experiment Station, 1941).

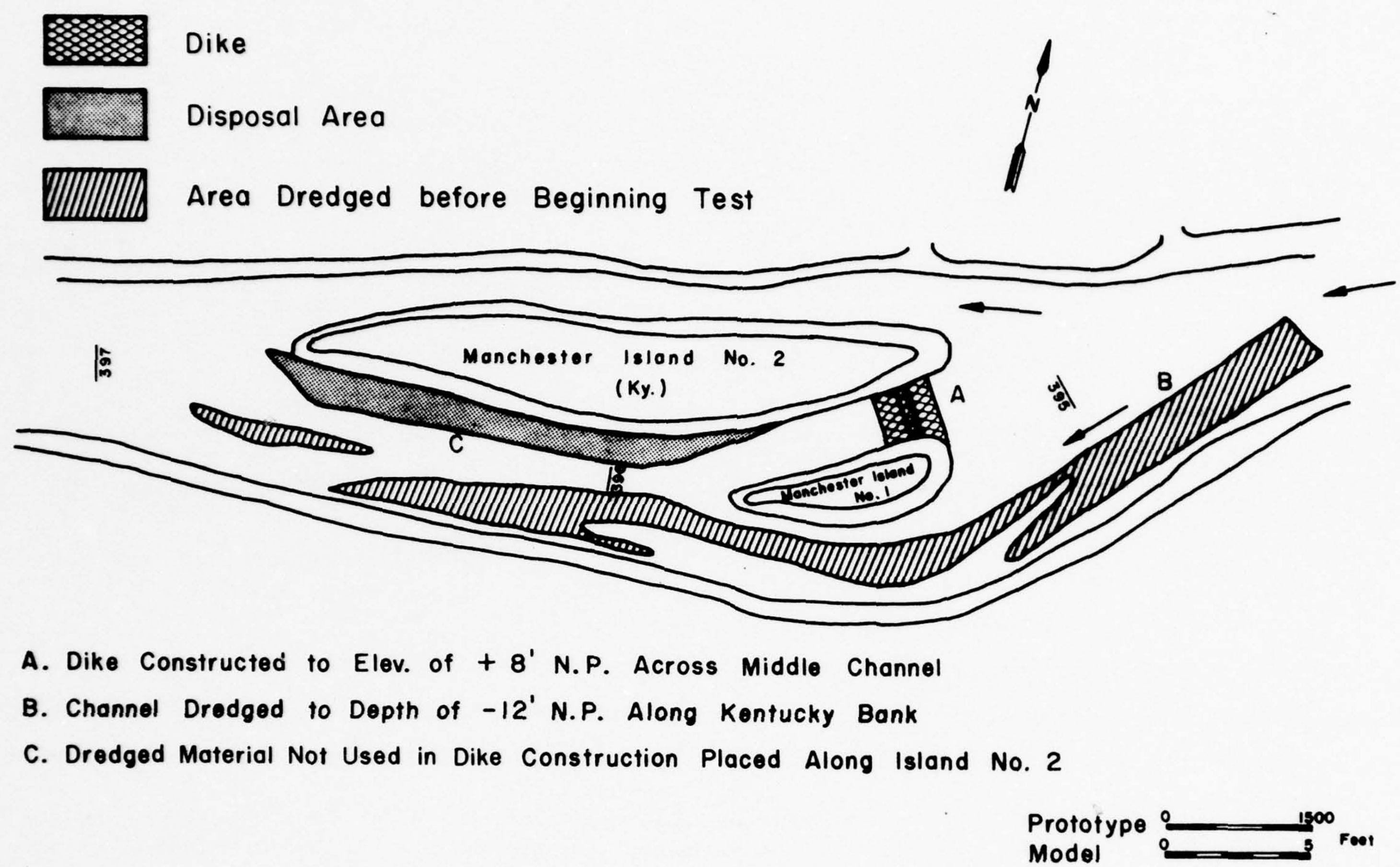
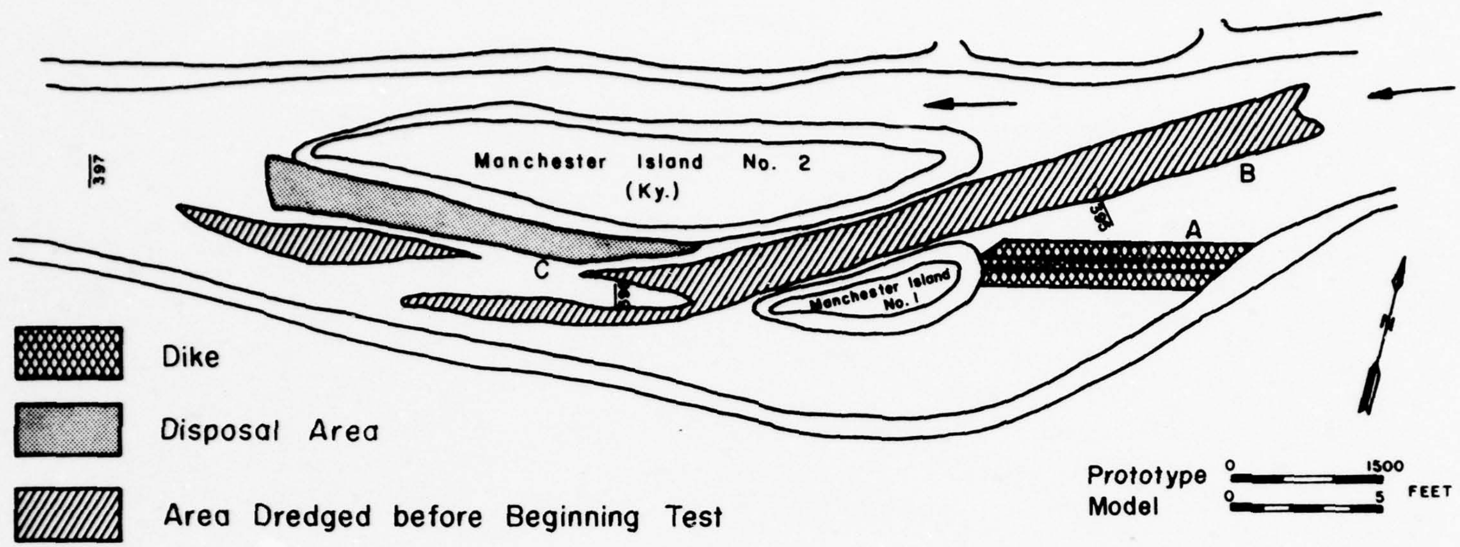


FIGURE 84. Plan A--Manchester Islands Model Study (after Corps of Engineers, Waterways Experiment Station, 1941).



A. Dike Constructed to Elev. of +8' N.P. Across Kentucky Channel

B. Channel Dredged to Depth of -12' N.P. Between Islands

C. Dredged Material Not Used in Dike Construction Placed Along Island No. 2

FIGURE 85. Plan B--Manchester Islands Model Study (after Corps of Engineers, Waterways Experiment Station, 1941).

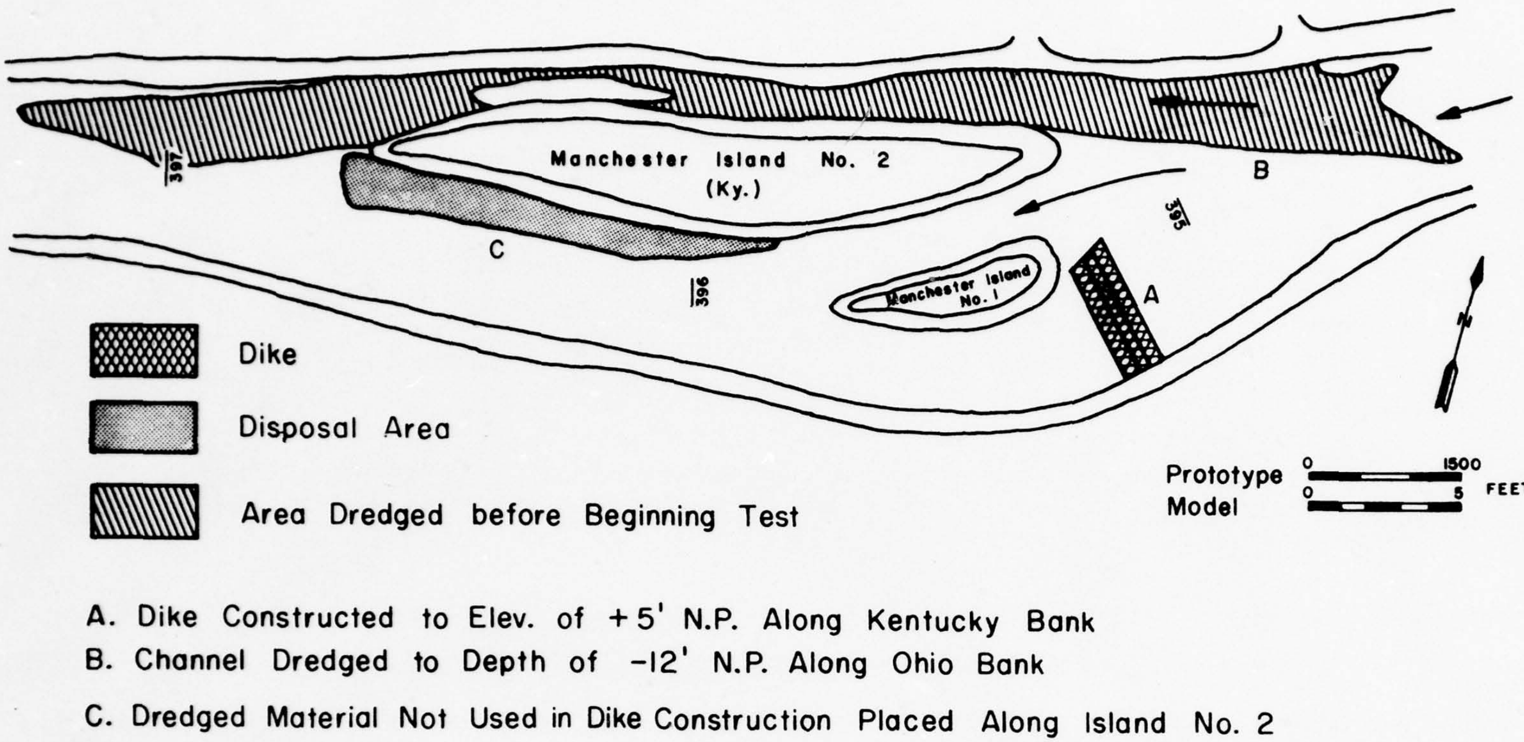


FIGURE 86. Plan C--Manchester Islands Model Study (after Corps of Engineers, Waterways Experiment Station, 1941).

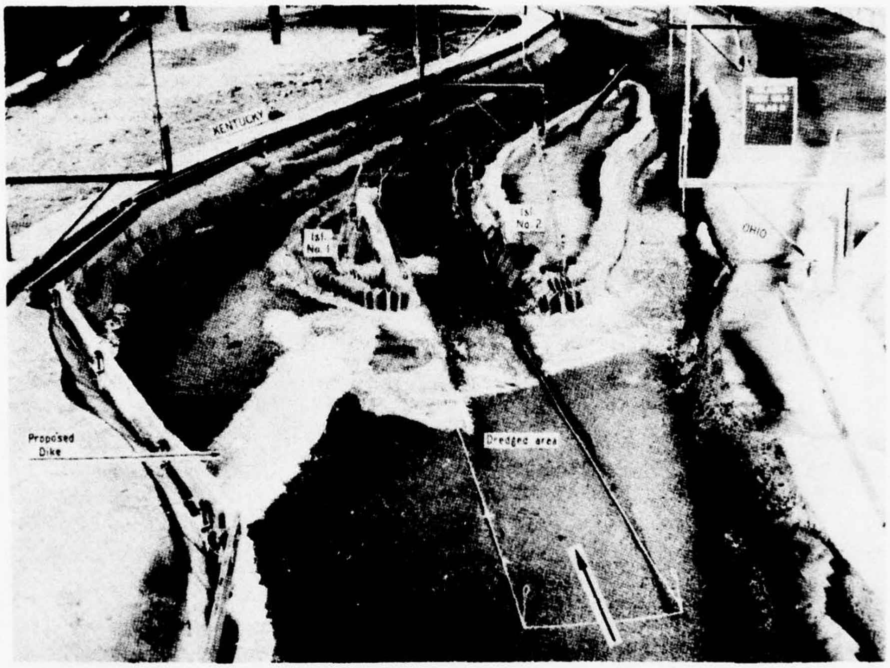


FIGURE 87. General Configuration of Manchester Islands Model Study. Location of Dike and Dredged Cut for Plan B (after Corps of Engineers, Waterways Experiment Station, 1941).

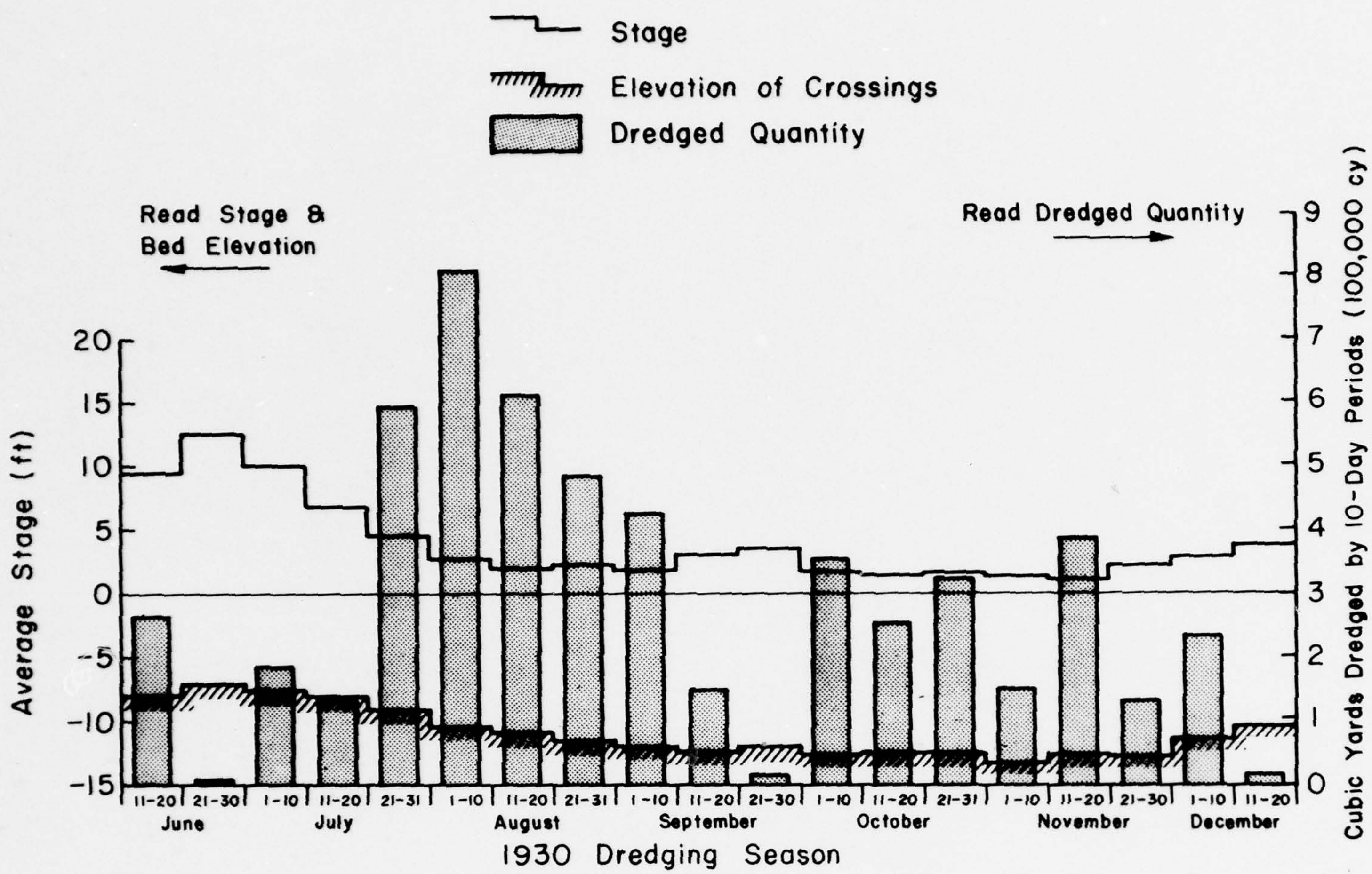


FIGURE 88. Relation of Stage to Depth on Crossings and Dredging Requirements.

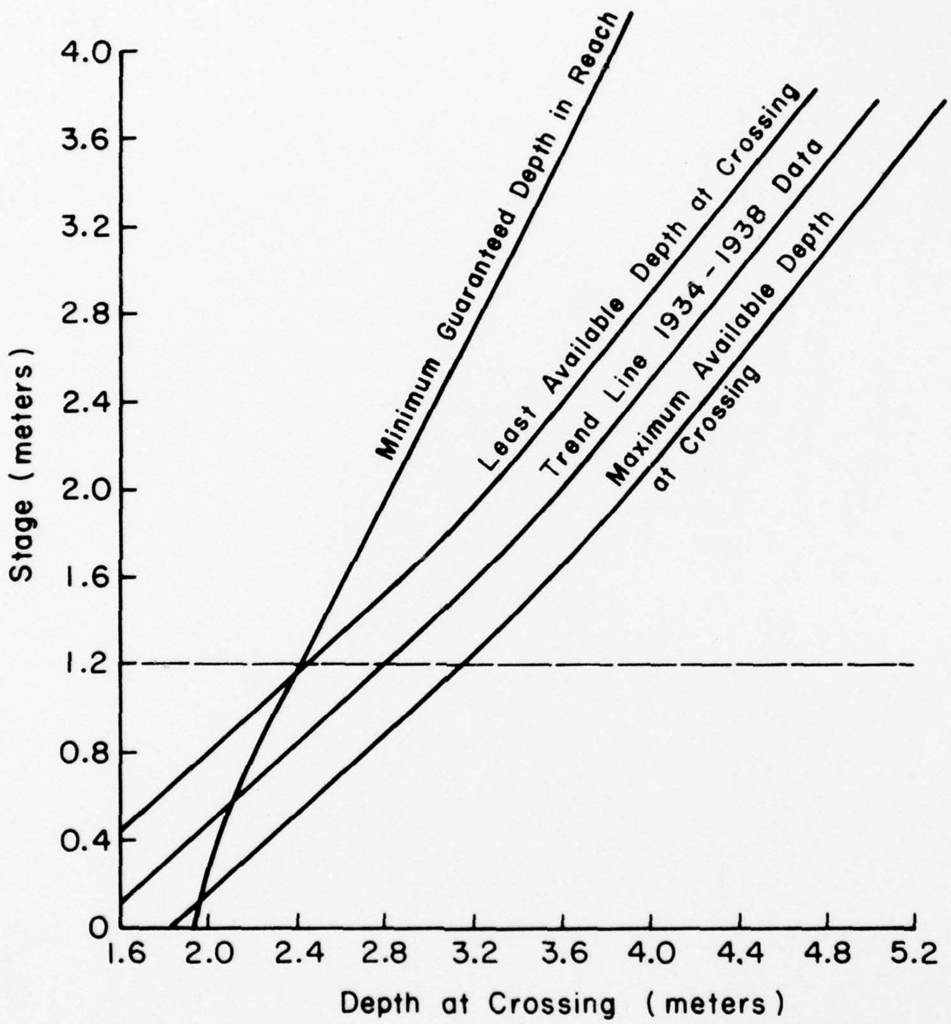


FIGURE 89. Dredging Requirements Related to Stage and Depth at Crossing (after Kondrat'ev et al., 1962).

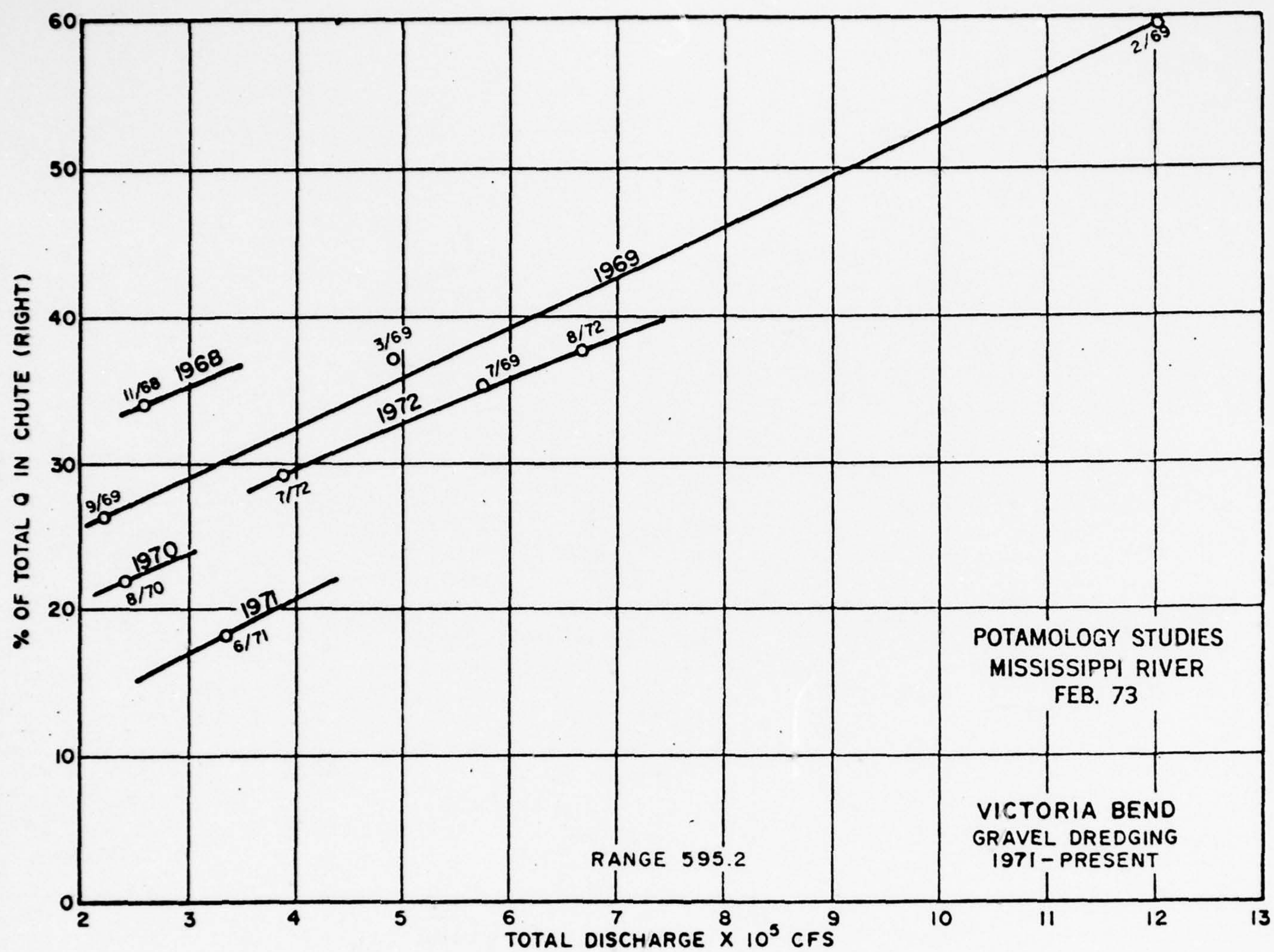


FIGURE 90. Victoria Bend Gravel Dredging 1971-Present (after Winkley and Harris, 1973).

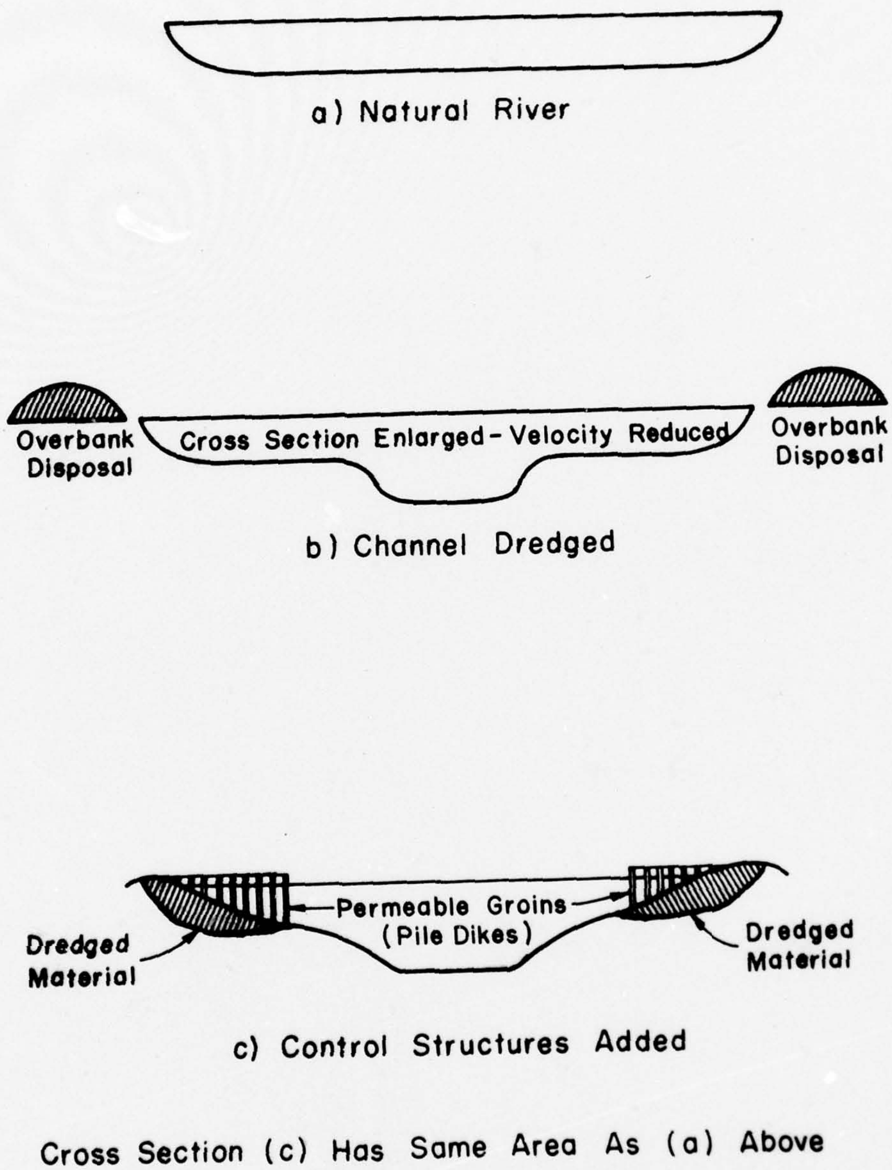


FIGURE 91. Schematic Cross Sections--Channel Dredging Concepts (after Kidby, 1966).

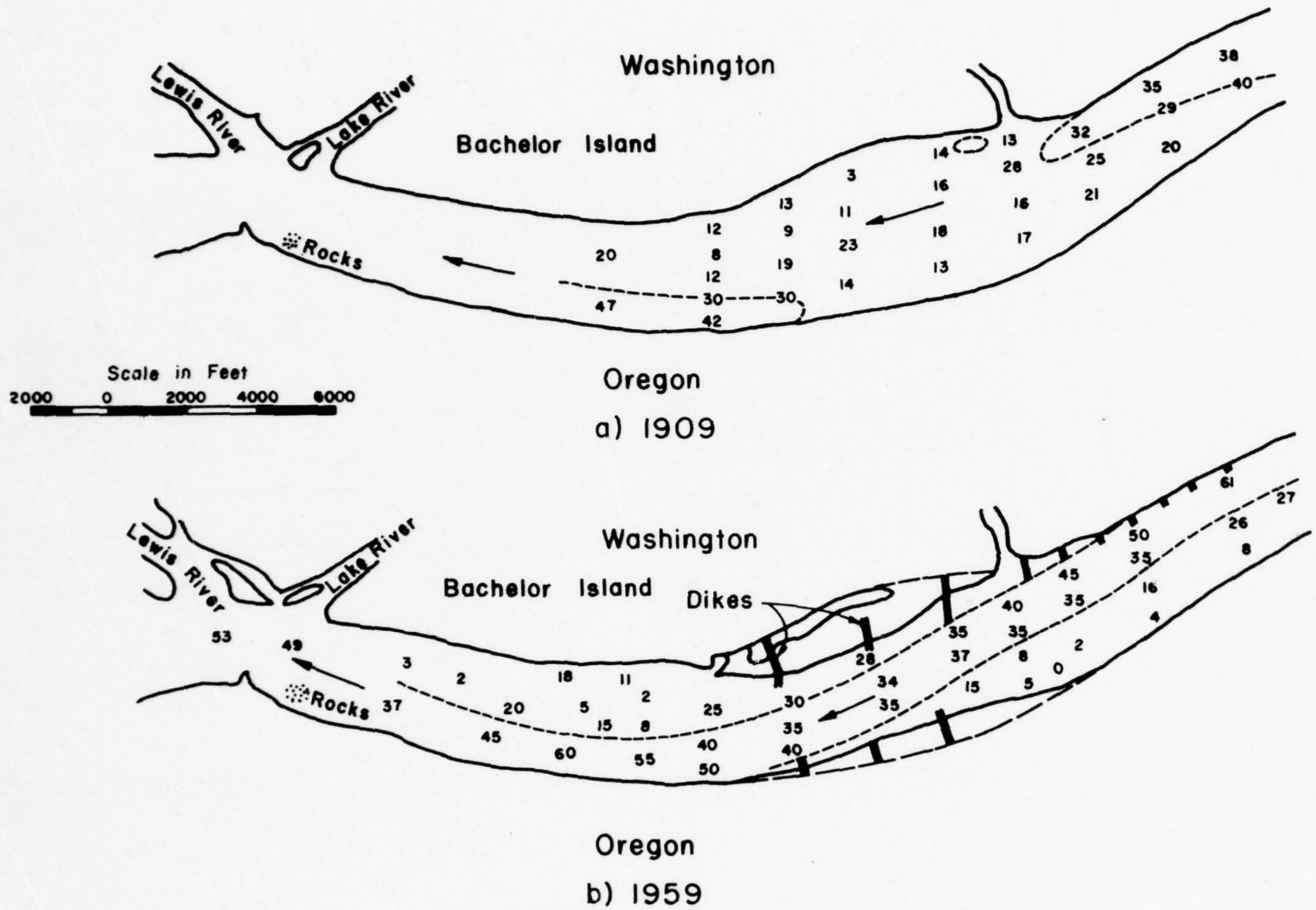


FIGURE 92. Henrici Bar, Columbia River (River Mile 91) 1909-1959 (after Hickson, 1965).

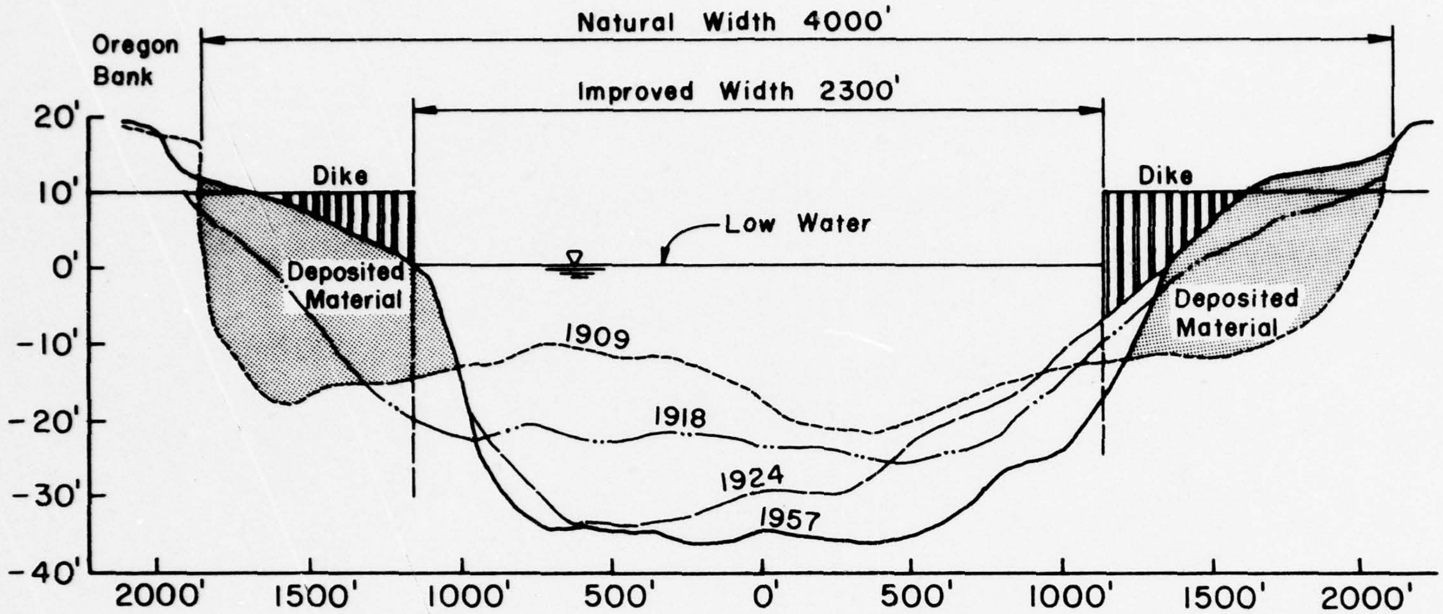


FIGURE 93. Cross Sections at Henrici Bar, Columbia River (River Mile 91) 1909-1959 (after Bubenik, 1963).

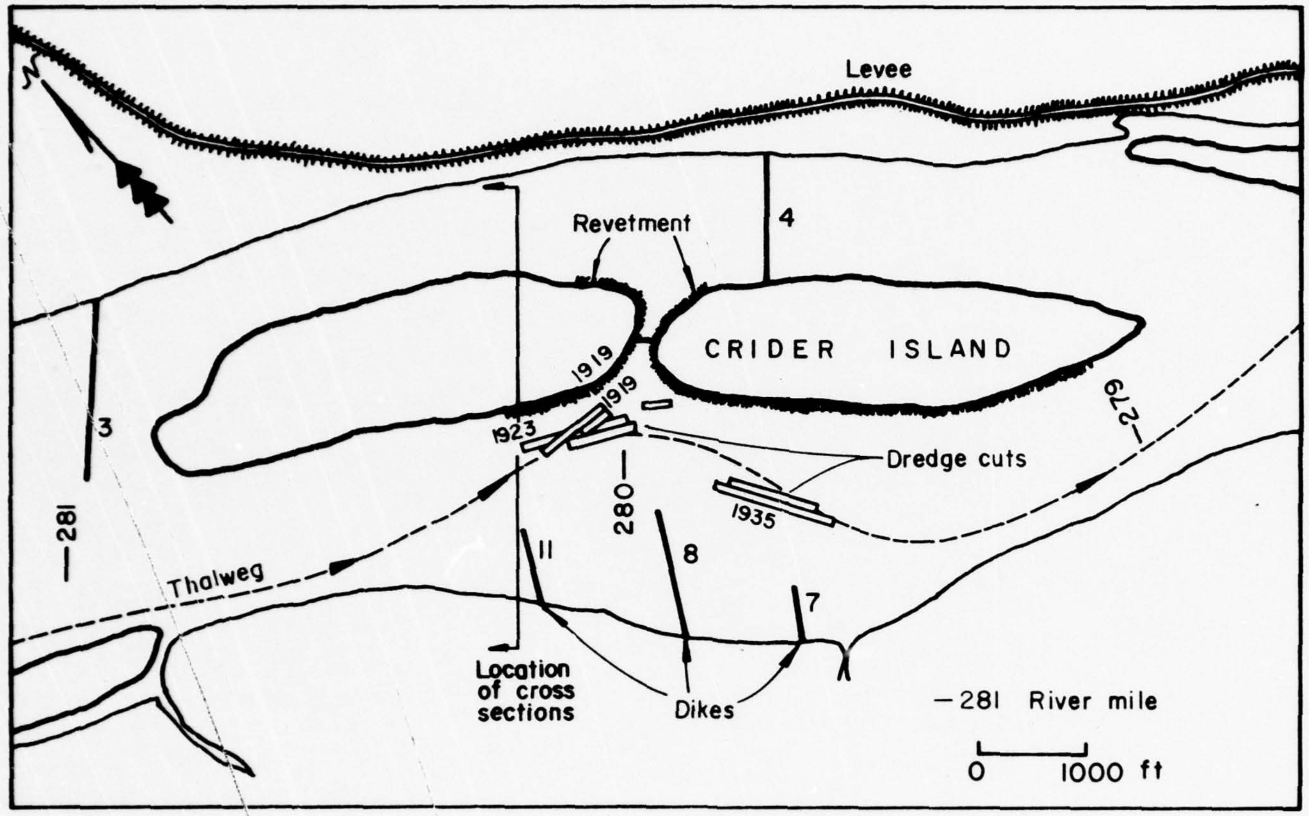


FIGURE 94. Map of Crider Island.

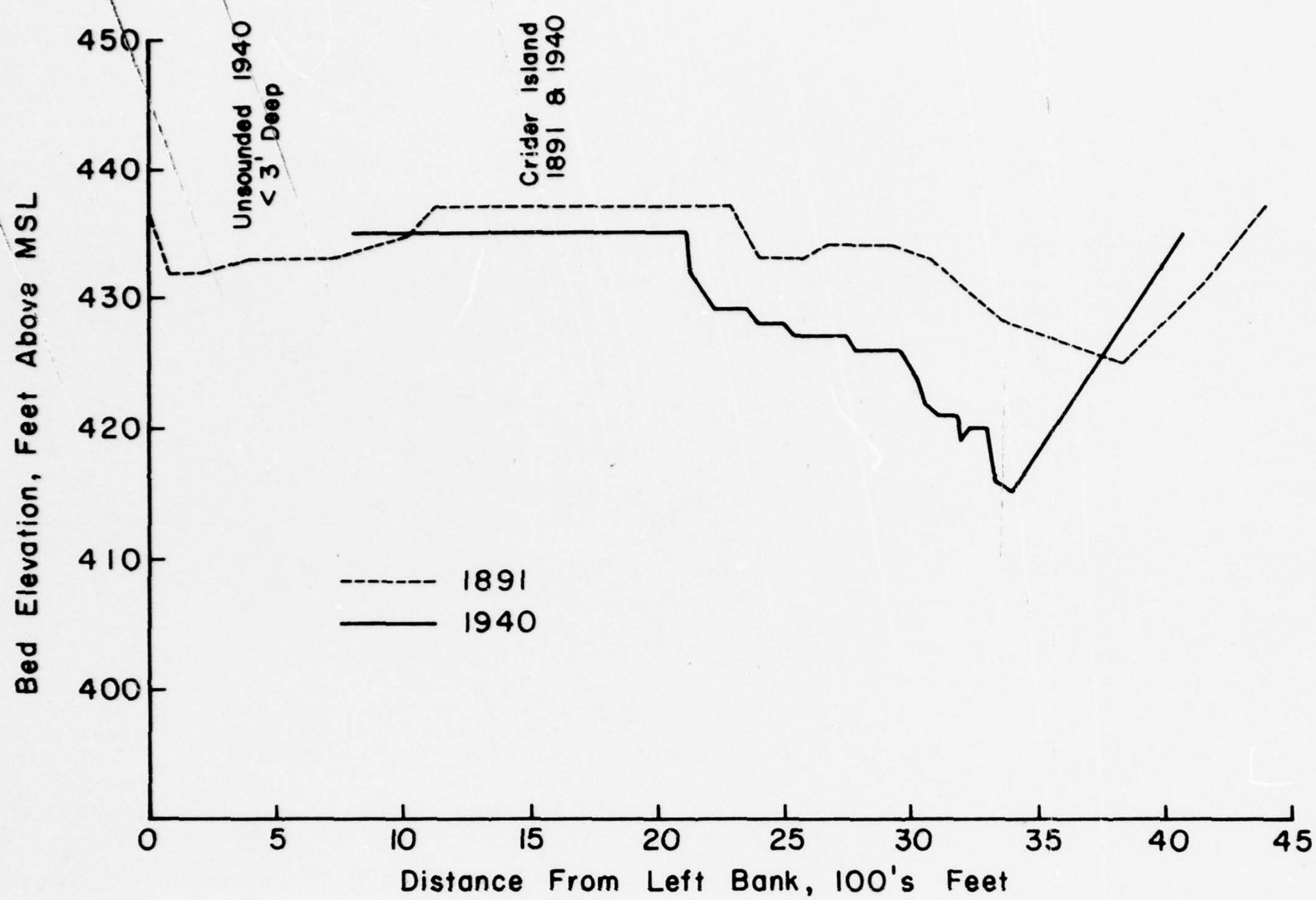


FIGURE 95. Crider Island Cross Section (RM 280).

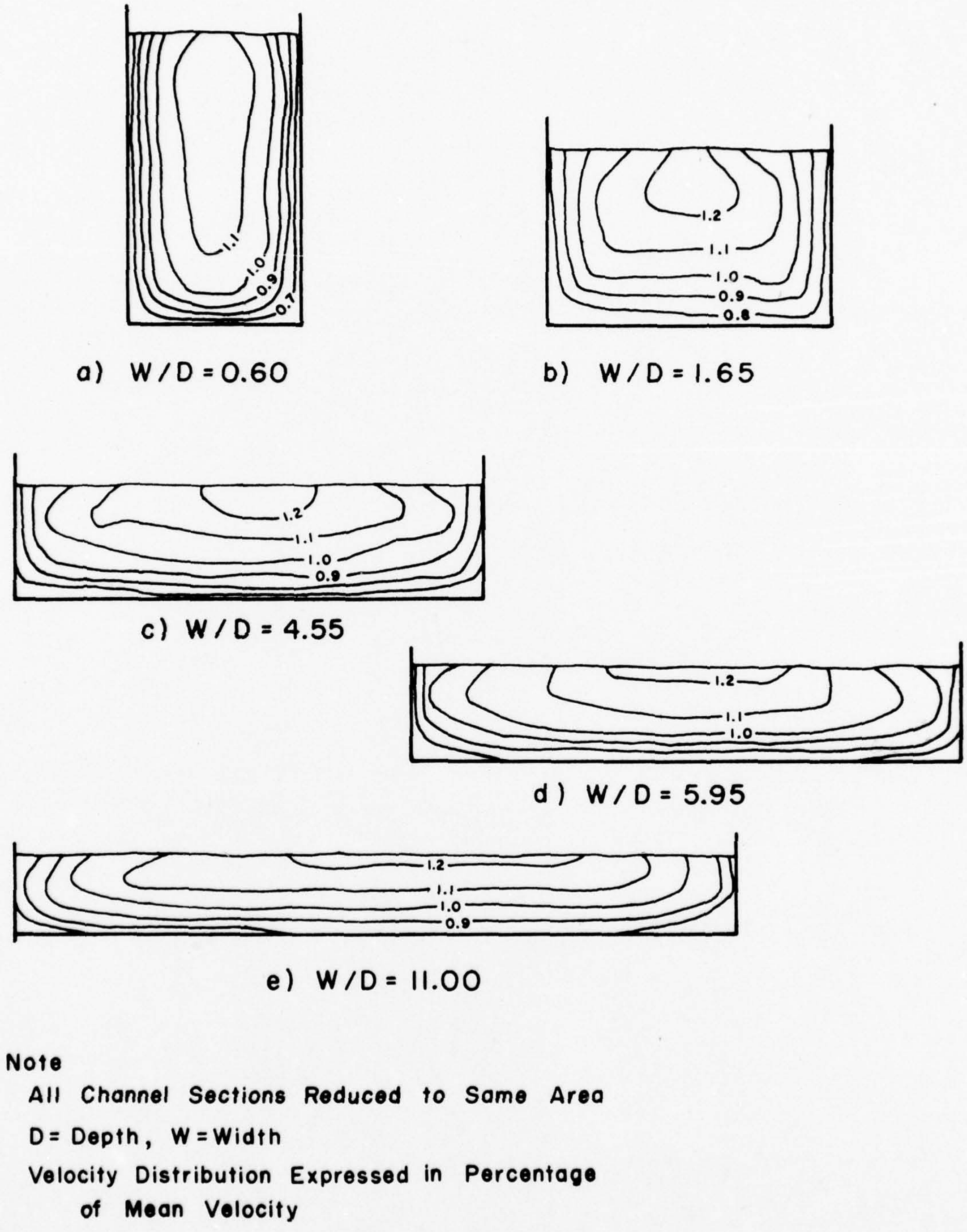


FIGURE 96. Relation of Velocity Distribution to Channel Shape in Rectangular Channels (after Lane, 1937).

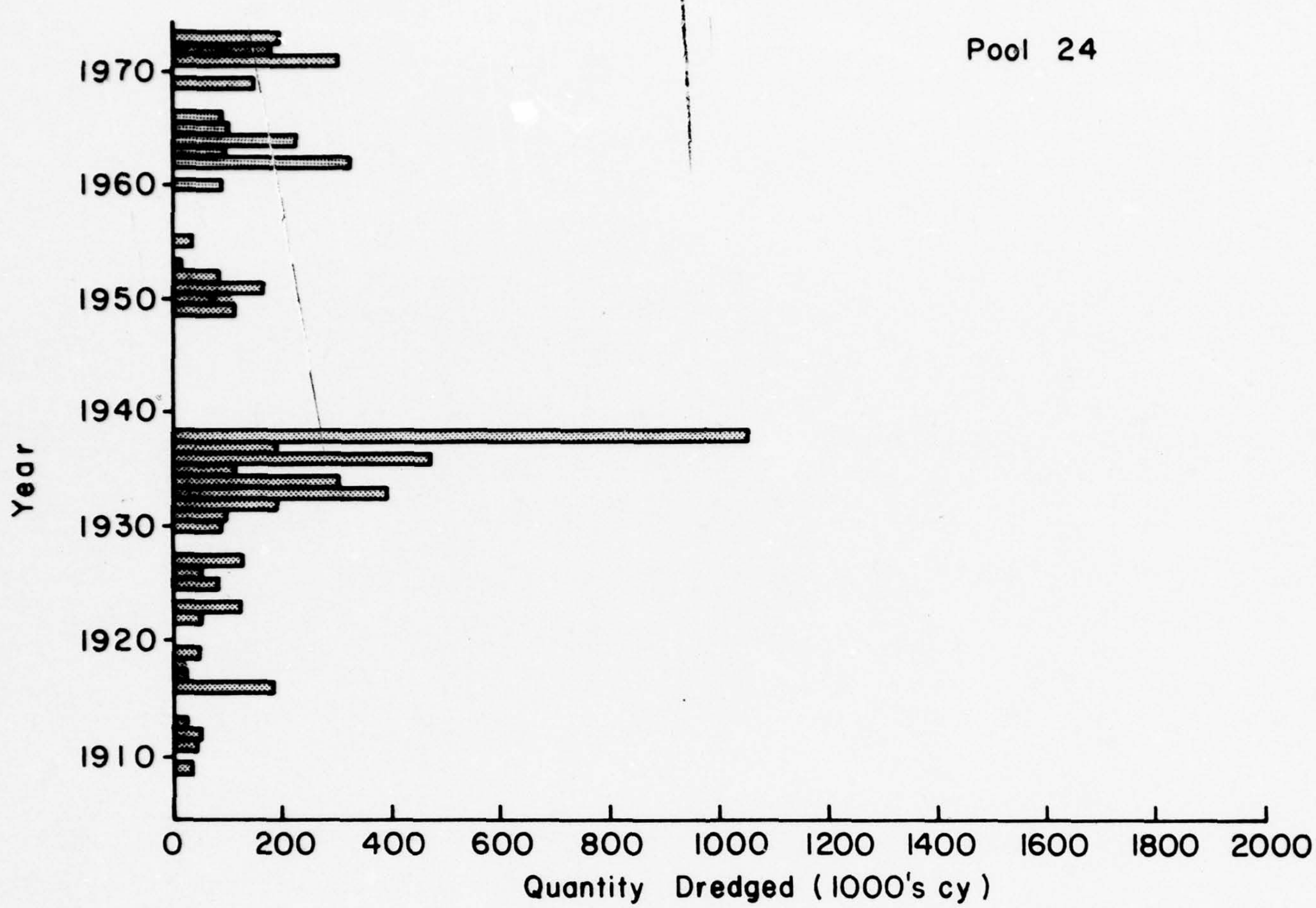


FIGURE 97. Dredged Volume by Year in Pool 24.

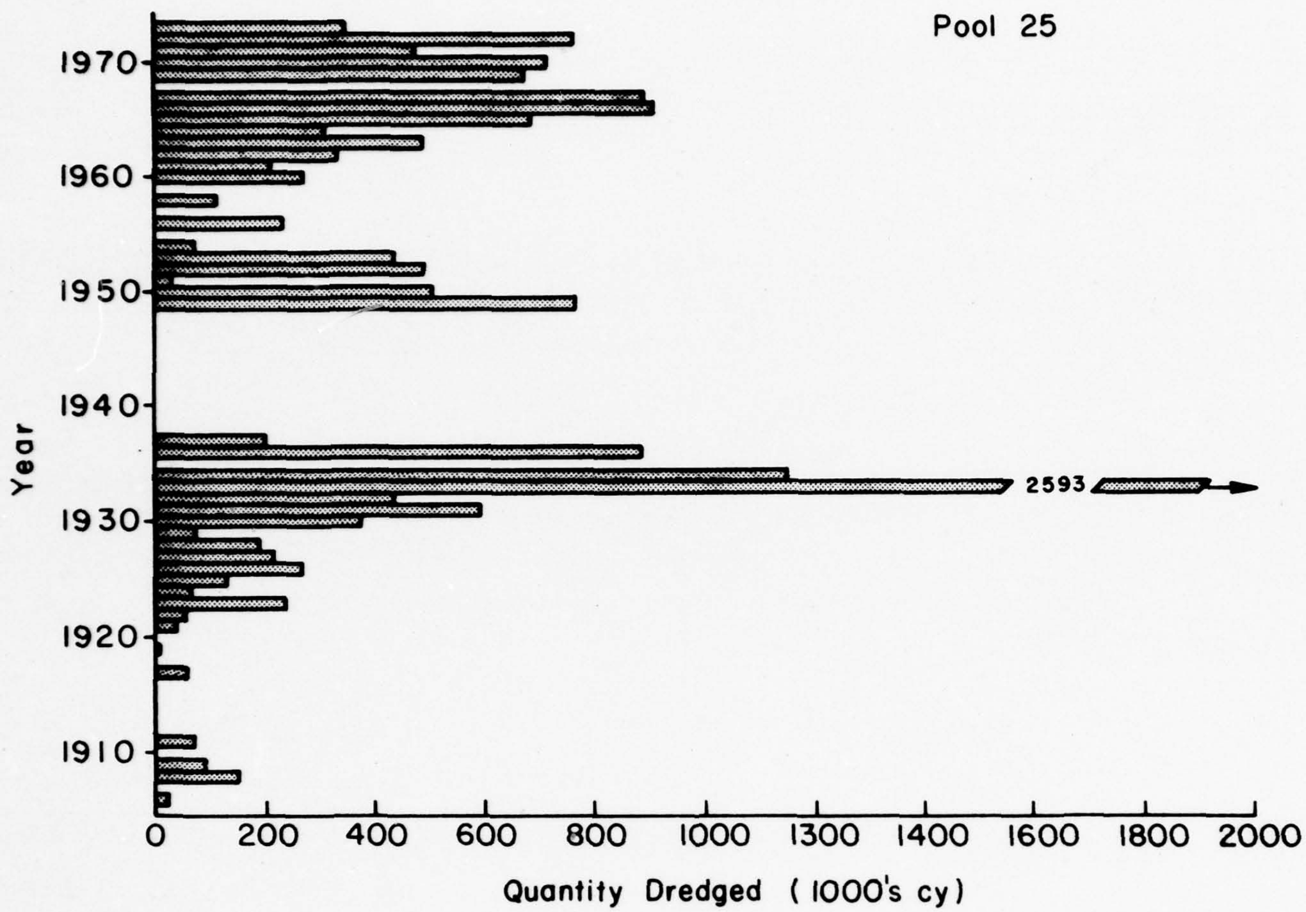


FIGURE 98. Dredged Volume by Year in Pool 25.

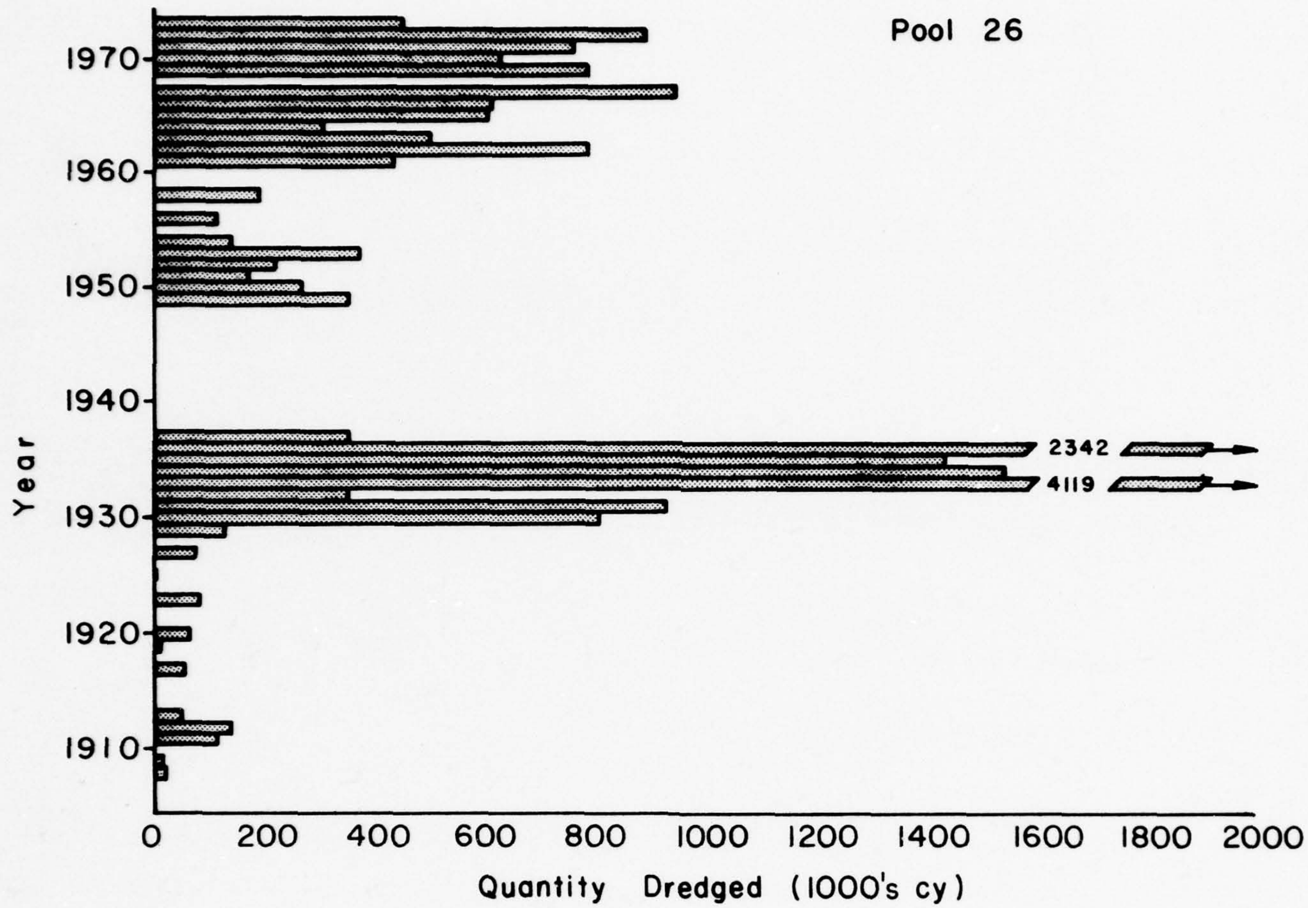


FIGURE 99. Dredged Volume by Year in Pool 26.

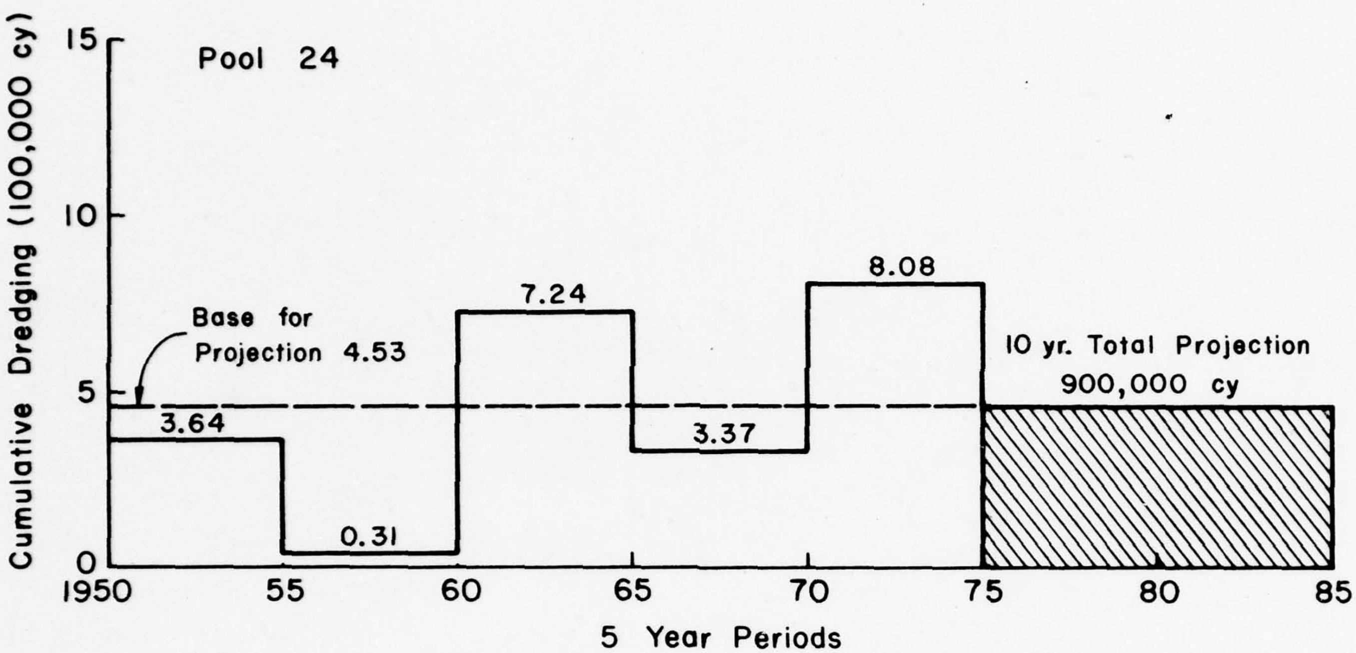


FIGURE 100. Five-Year Dredging Summary and Ten-Year Projection--Pool 24.

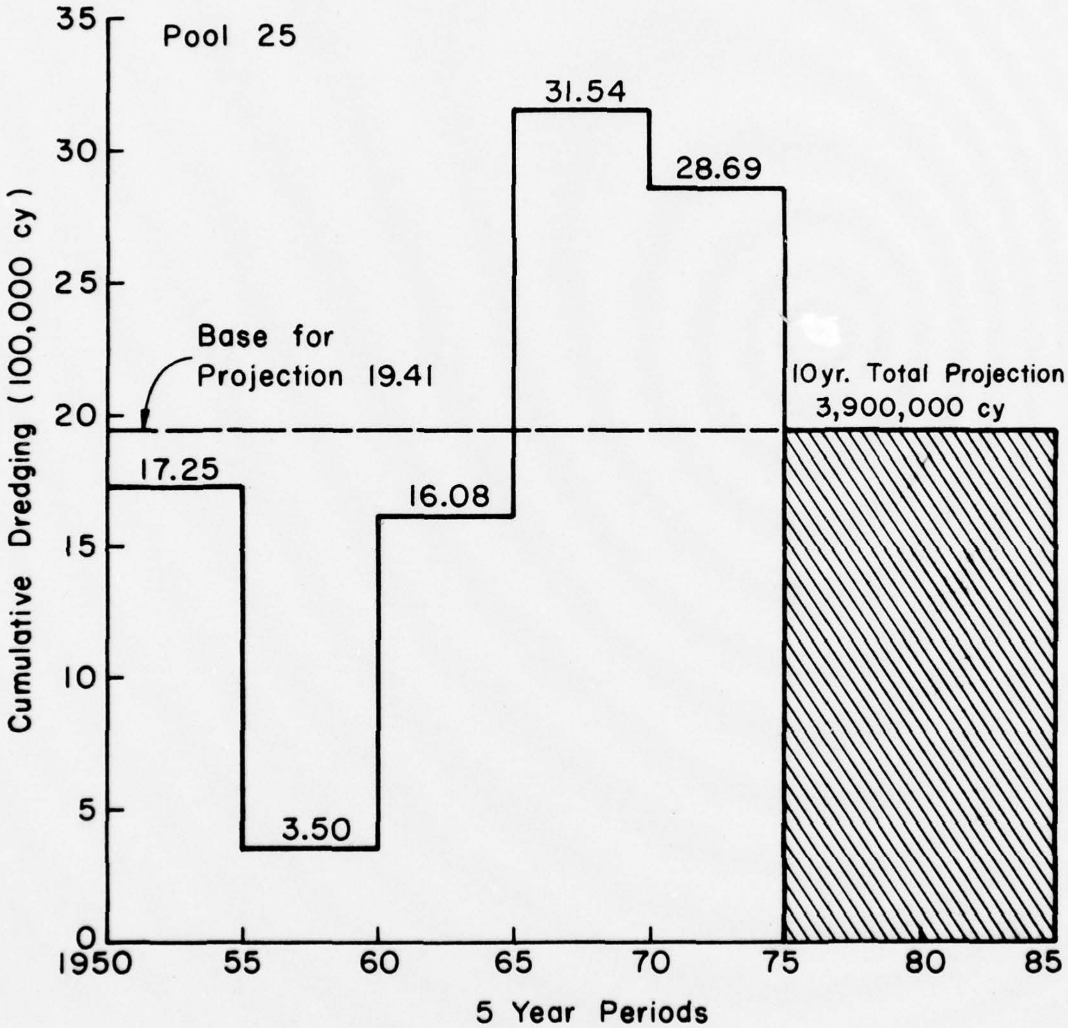


FIGURE 101. Five-Year Dredging Summary and Ten-Year Projection--Pool 25.

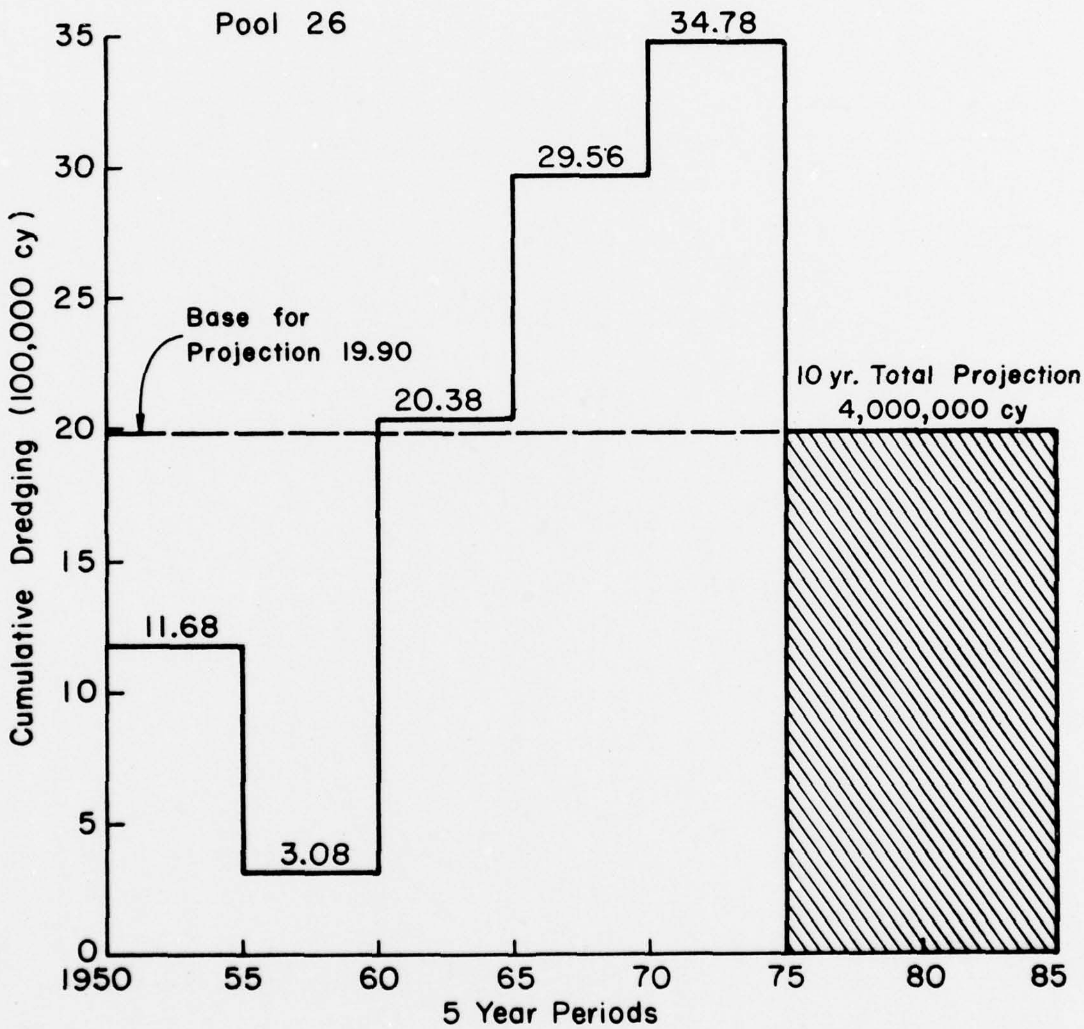


FIGURE 102. Five-Year Dredging Summary and Ten-Year Projection--Pool 26.

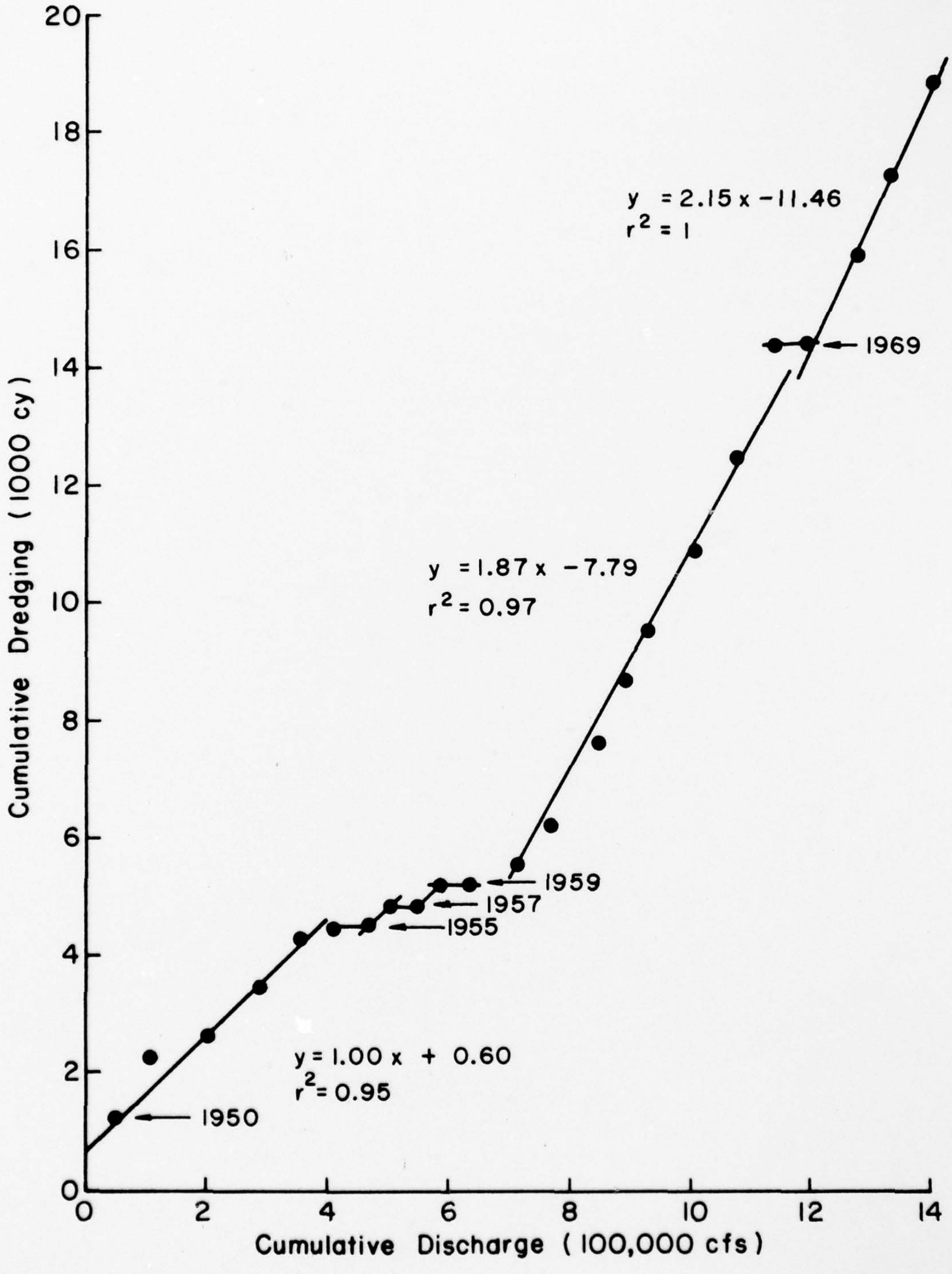


FIGURE 103. Cumulative Annual Dredging in Pools 24, 25, and 26 versus Cumulative Mean Annual Discharge at Keokuk, Iowa.

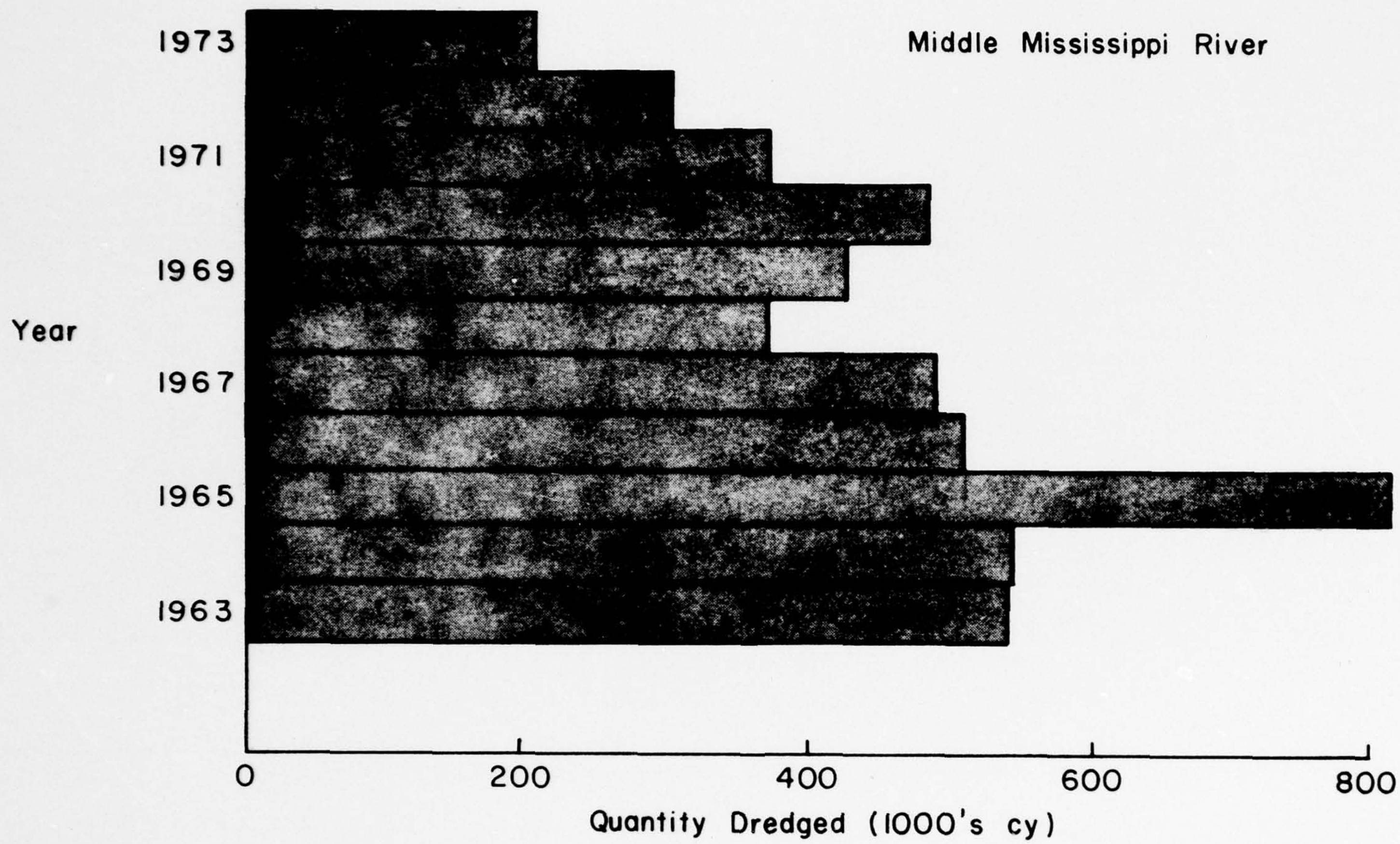


FIGURE 104. Dredged Volume by Year in Middle Mississippi River.

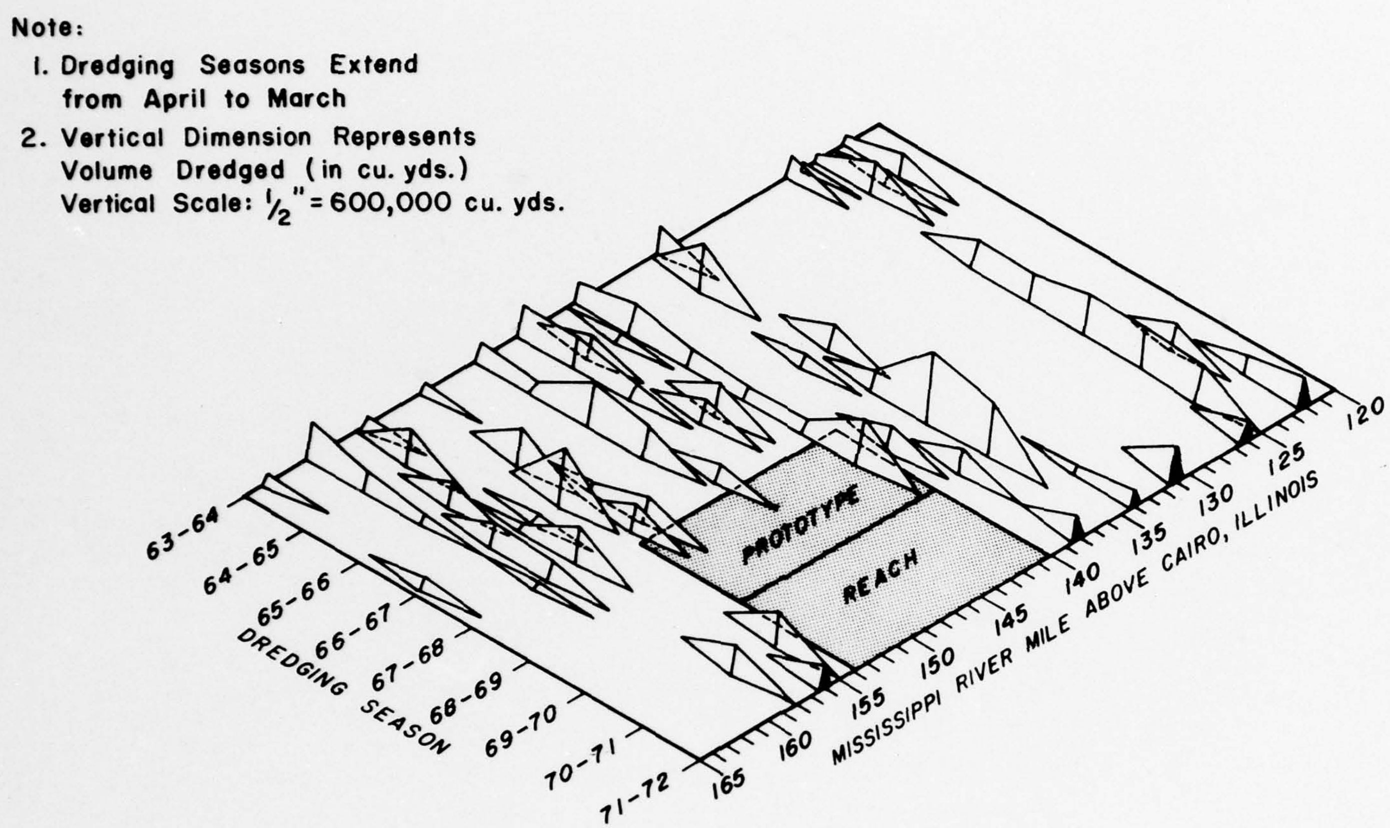


FIGURE 105. Isometric Drawing of Dredging History, RM 165 to RM 120, Middle Mississippi River (after Degenhardt, 1973).

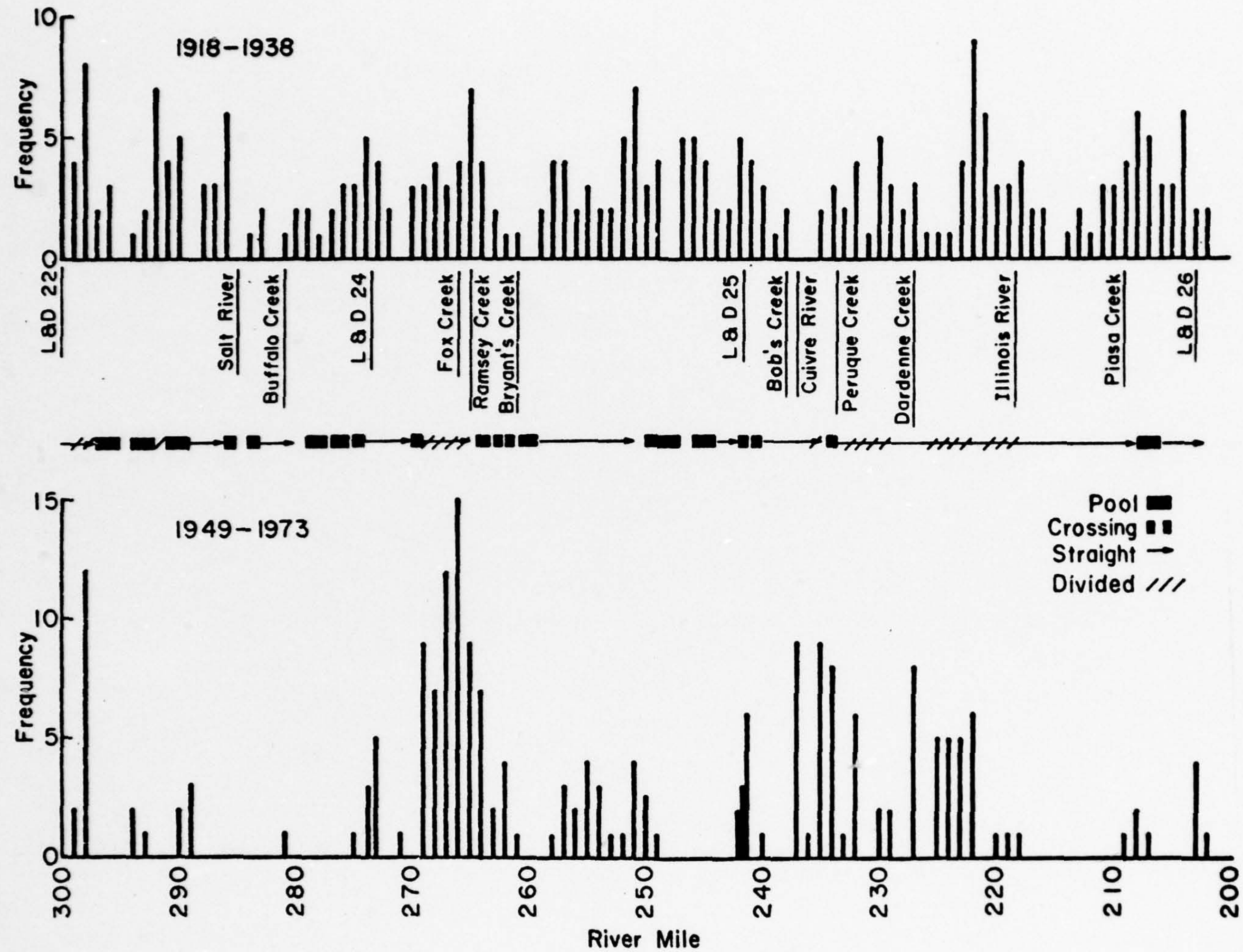


FIGURE 106. Dredging Frequency by Location in Pools 24, 25, and 26, Upper Mississippi River.

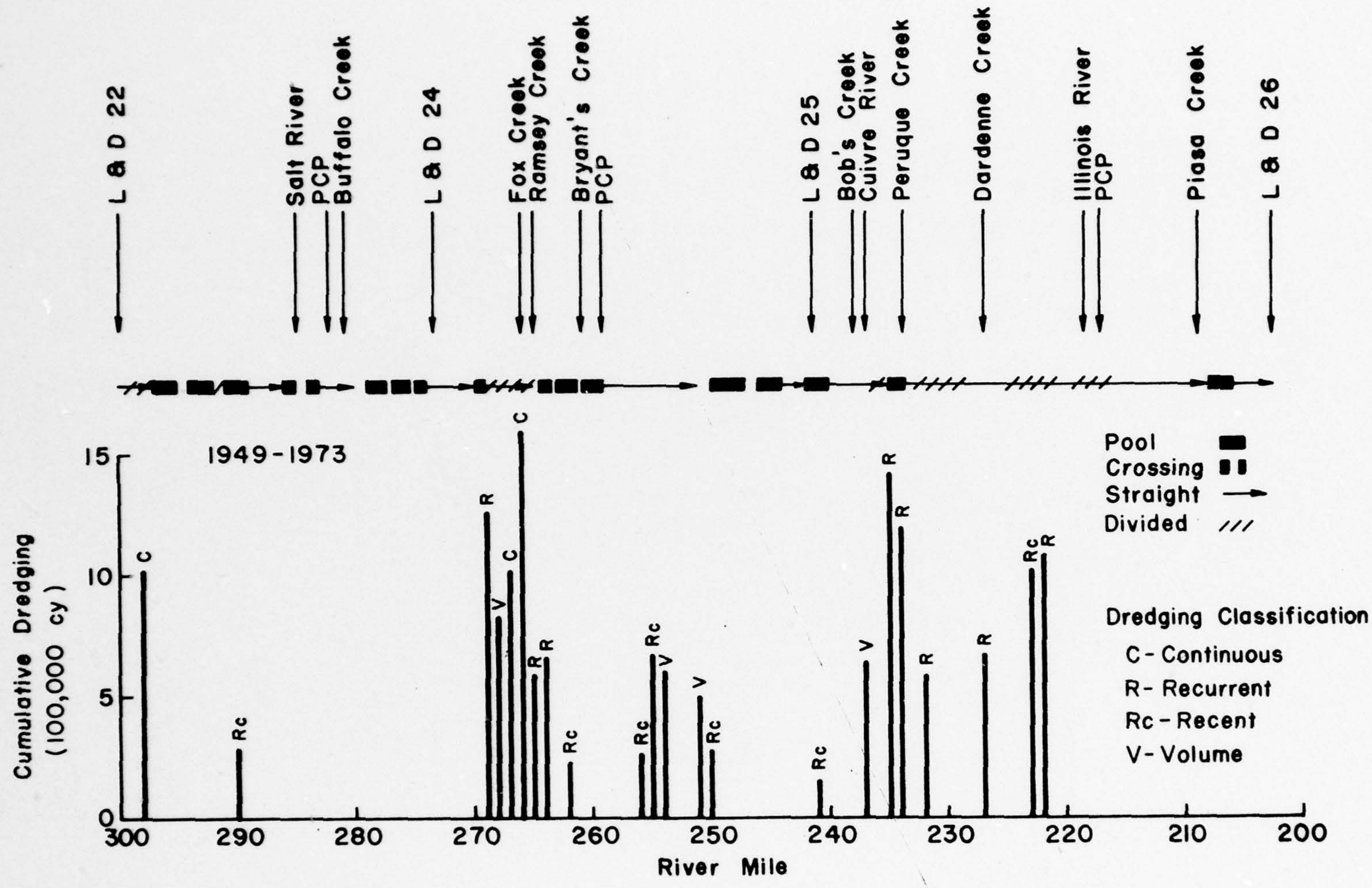


FIGURE 107. Location of Continuous, Recurrent, Recent, and Volume Dredging Problem Areas in Pools 24, 25, and 26, Upper Mississippi River.

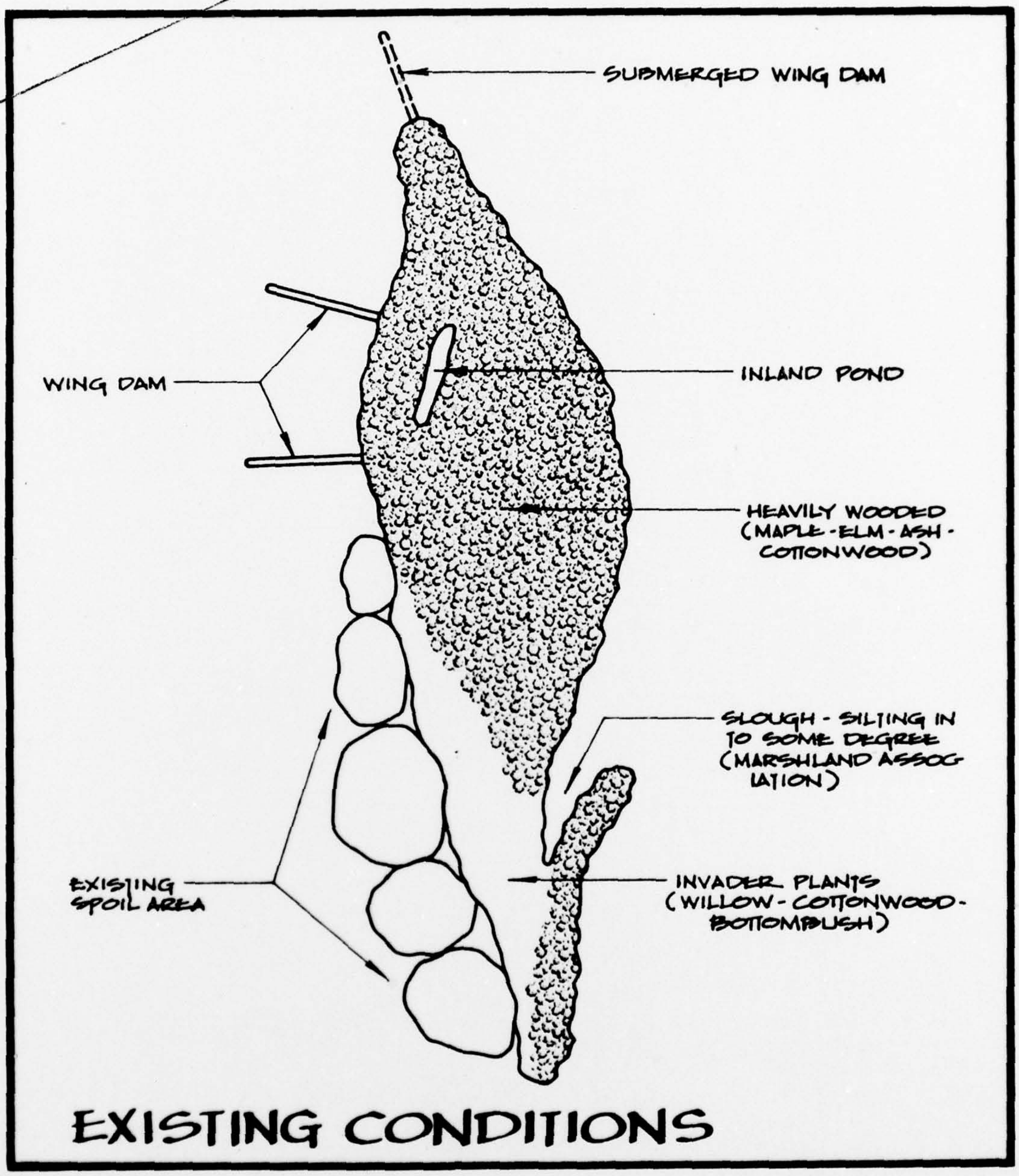


FIGURE 108. Sketch of Existing Conditions at Bass Island (after Corps of Engineers, Rock Island, 1974).

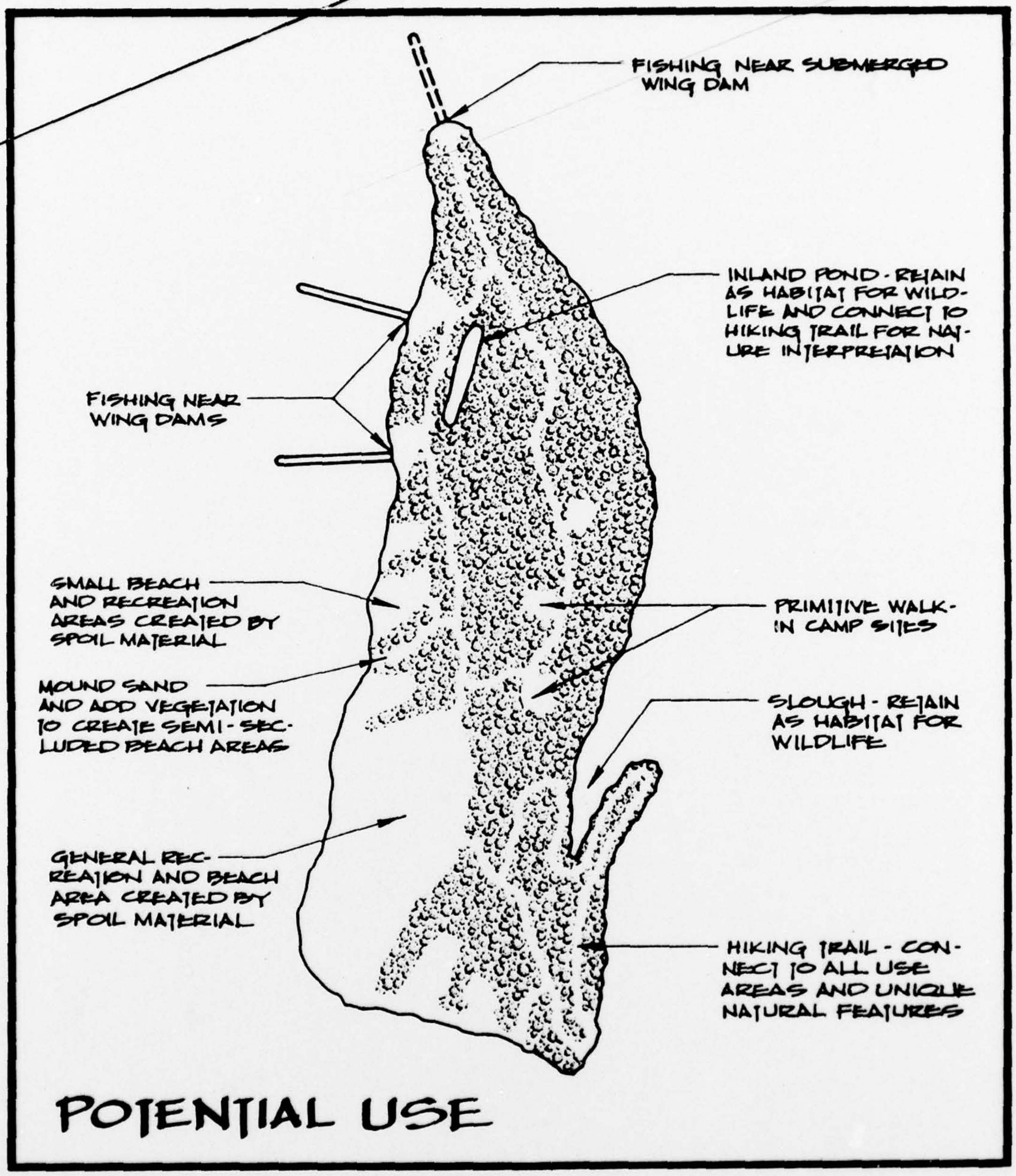


FIGURE 109. Sketch of Potential Use of Dredged Material for Recreational Enhancement of Bass Island (after Corps of Engineers, Rock Island, 1974).

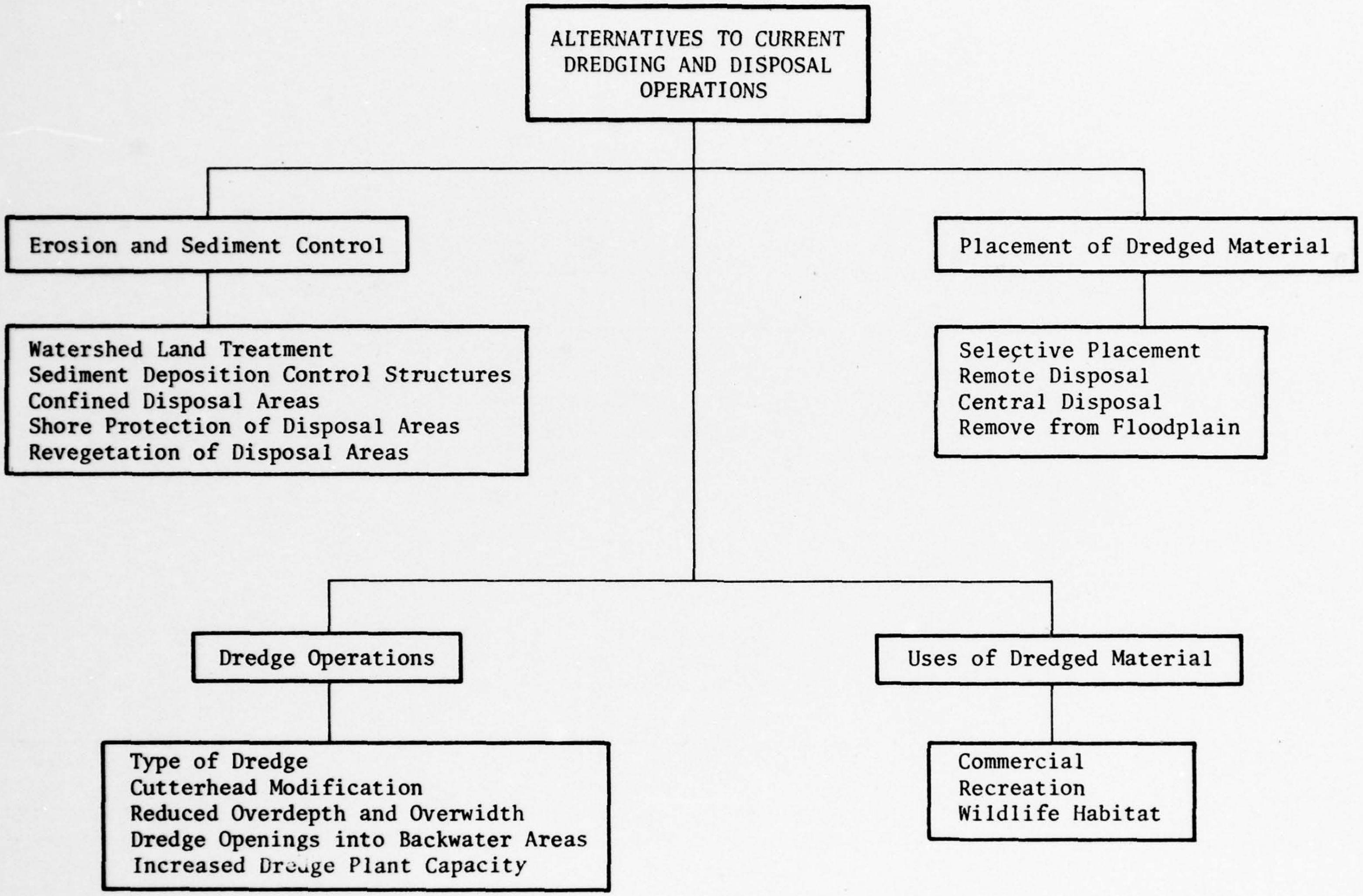


FIGURE 110. Alternatives to Current Dredging and Disposal Operations

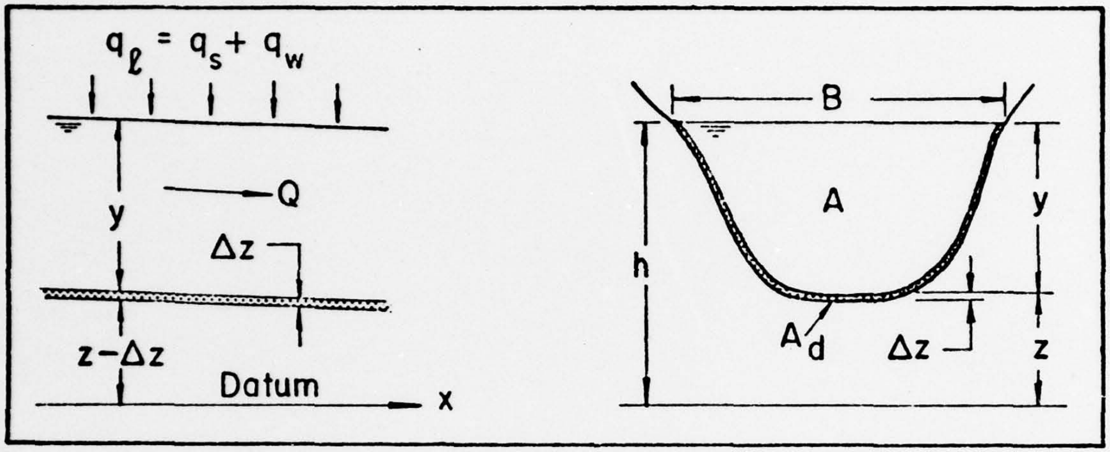


FIGURE 111. Definition Sketch of an Alluvial Channel (after Simons et al., 1975).

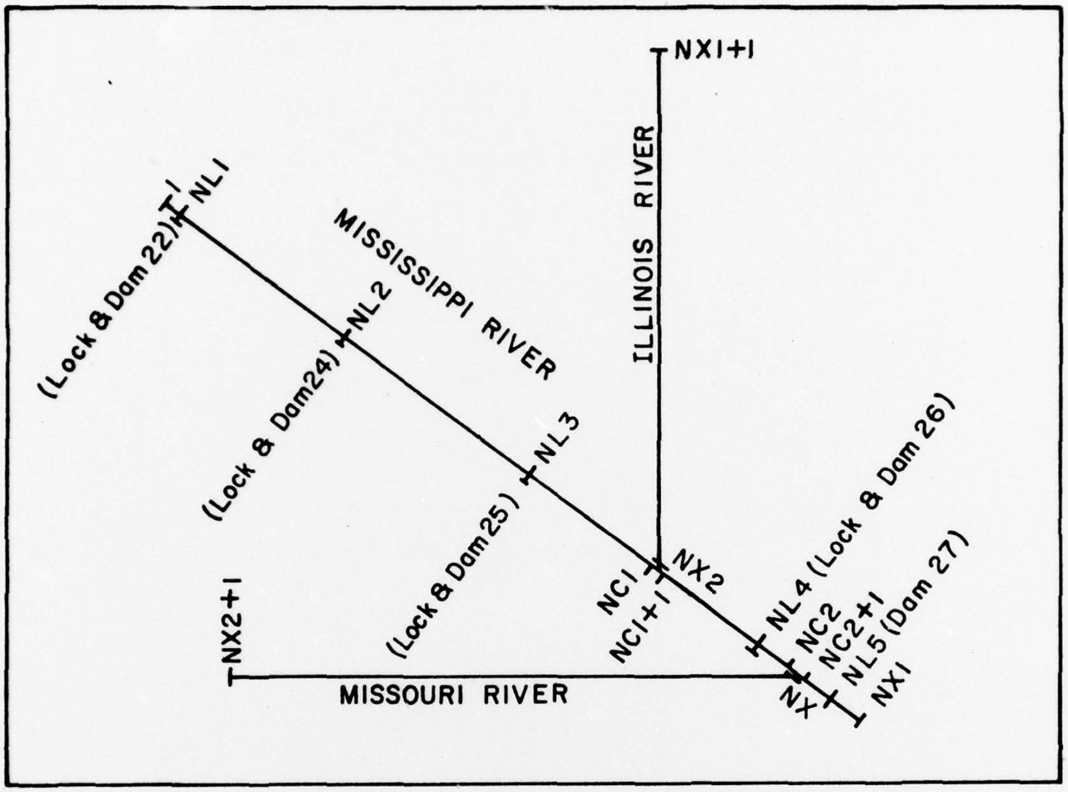


FIGURE 112. Schematic Diagram of the Study River Reach (after Simons et al., 1975).

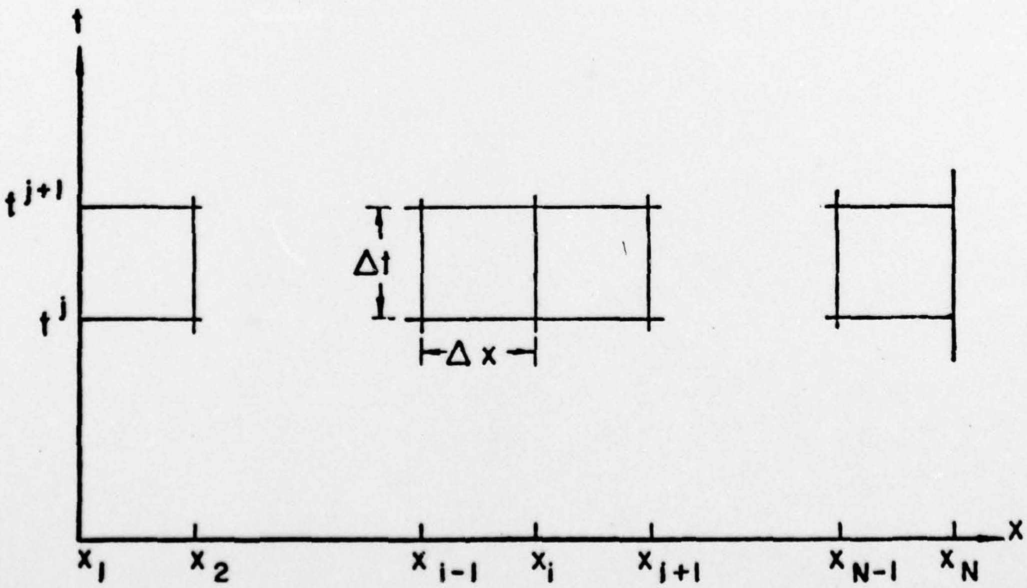


FIGURE 113. Network for the Implicit Method.

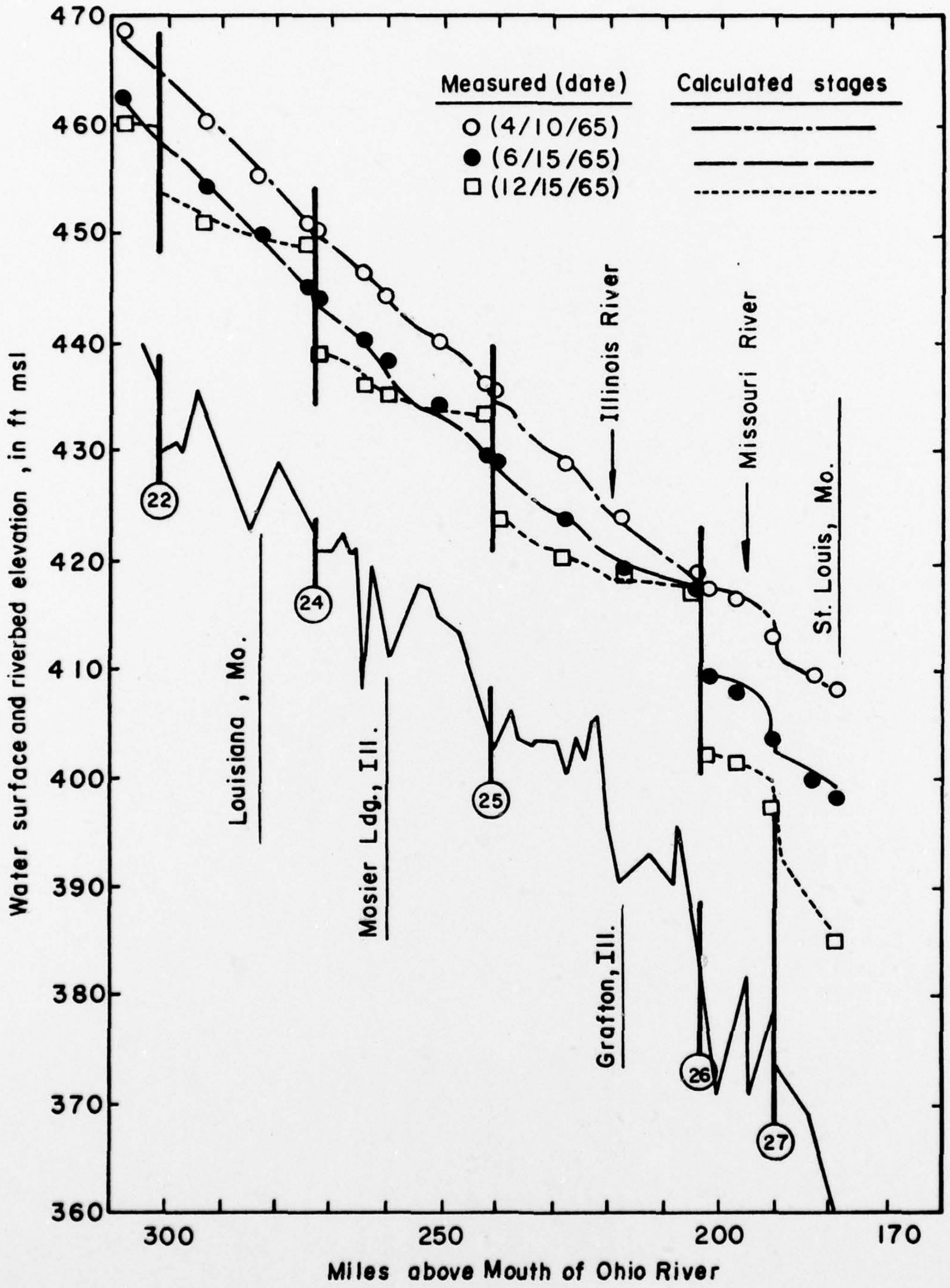


FIGURE 114. Mathematical Reproduction of 1965 Water Surface Profile in the Upper Mississippi River (after Simons et al., 1975).

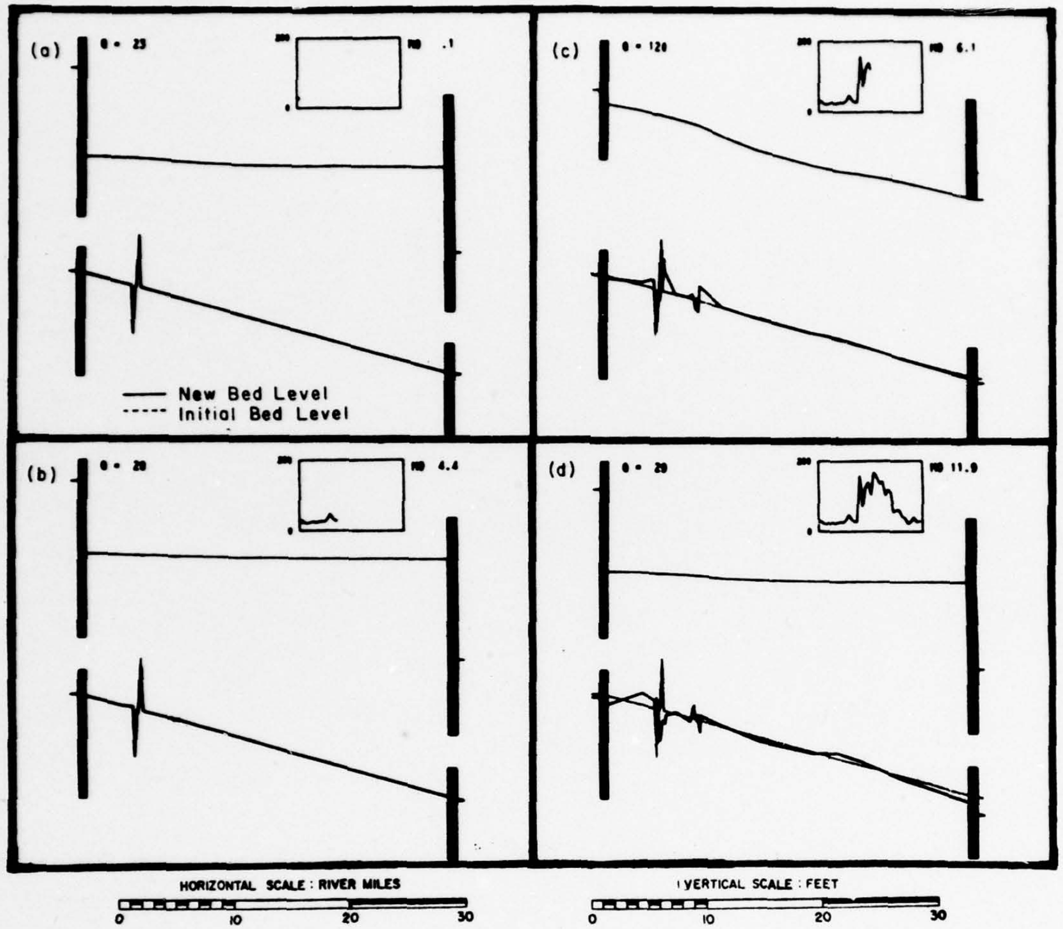


FIGURE 115. Riverbed Level Changes during the Year after Dredging and Disposing in a Downstream Pool (after Simons et al., 1975).

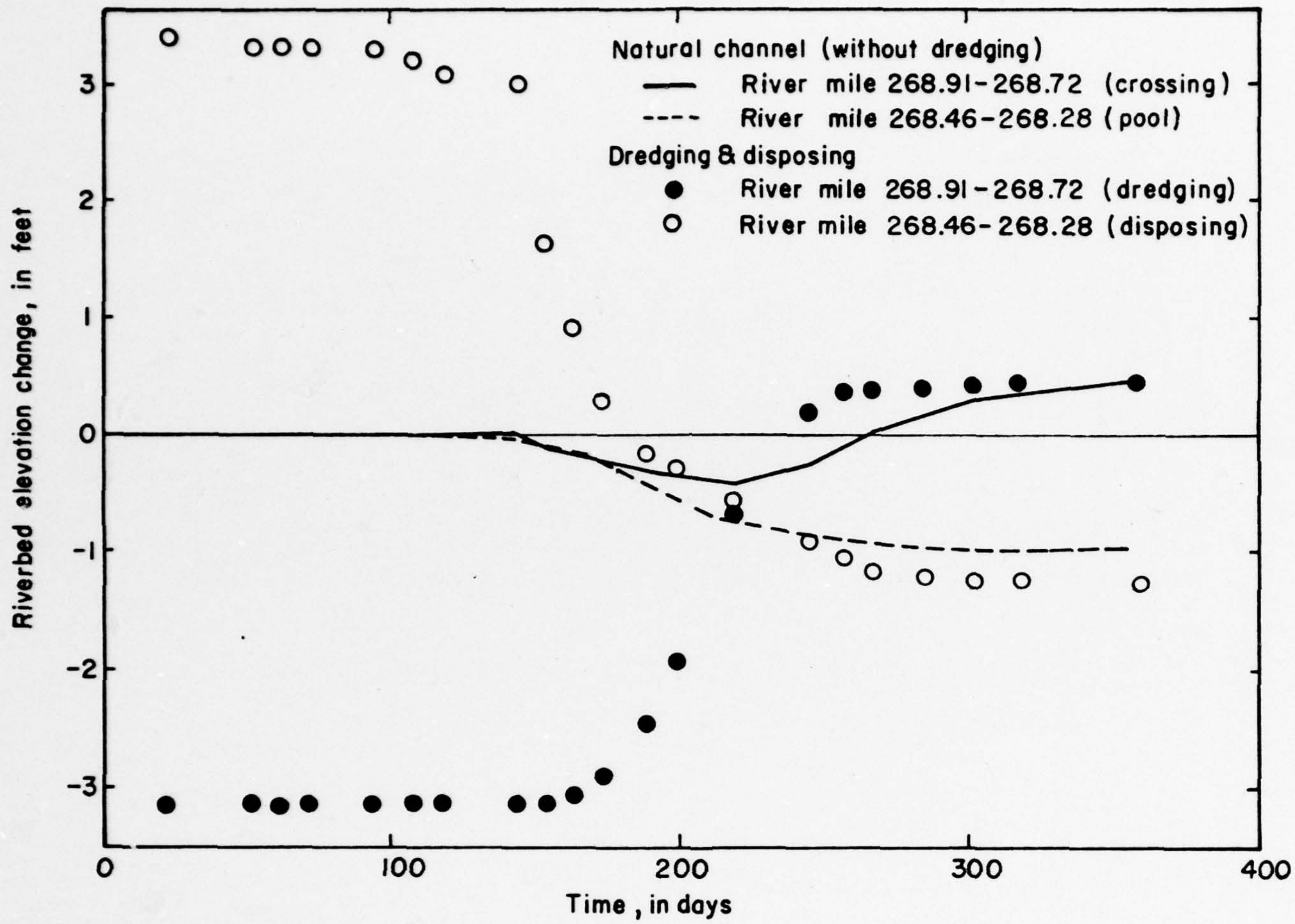
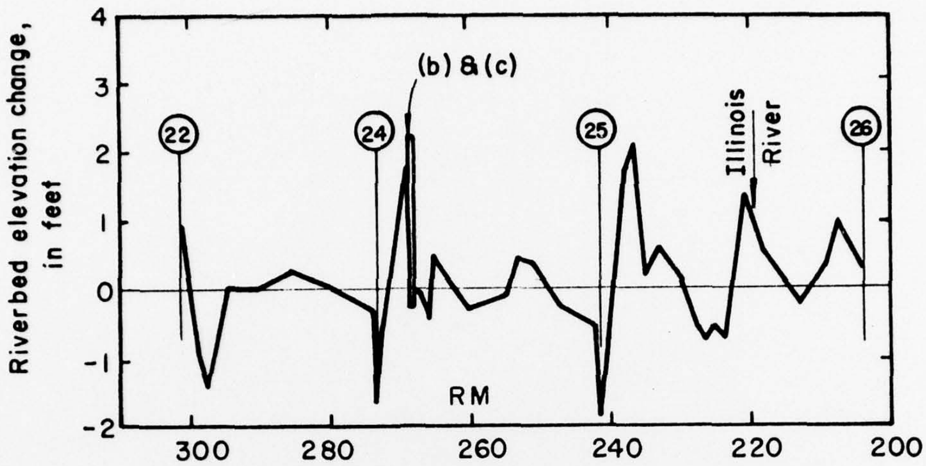
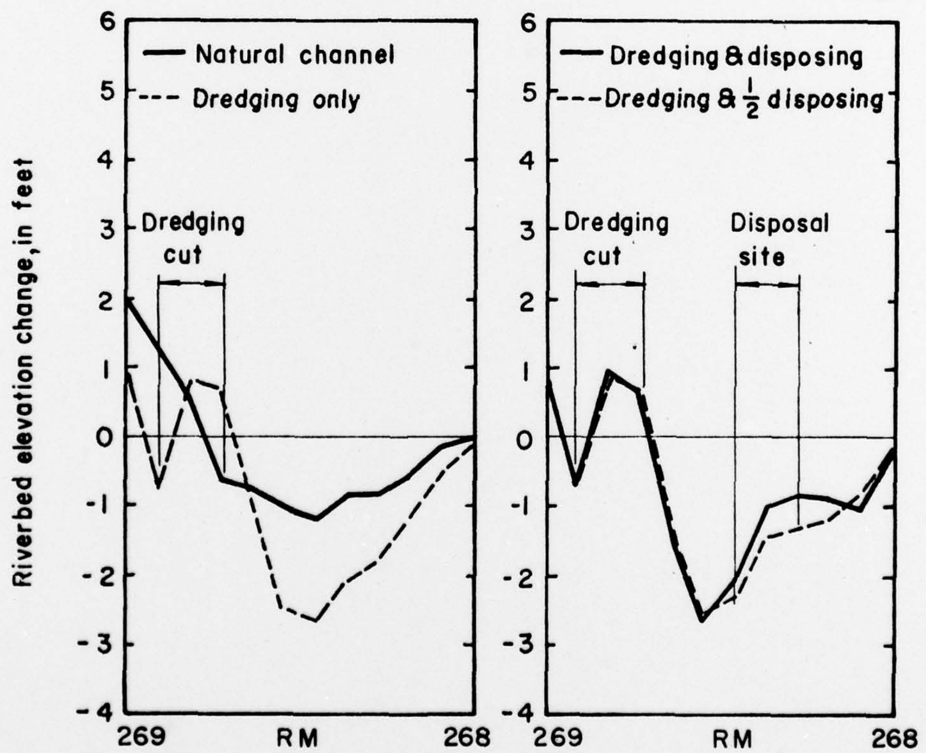


FIGURE 116. Average Riverbed Elevation Changes with Time in a Crossing and Its Downstream Pool Area (2-Year Annual Hydrograph) (after Simons et al., 1975).



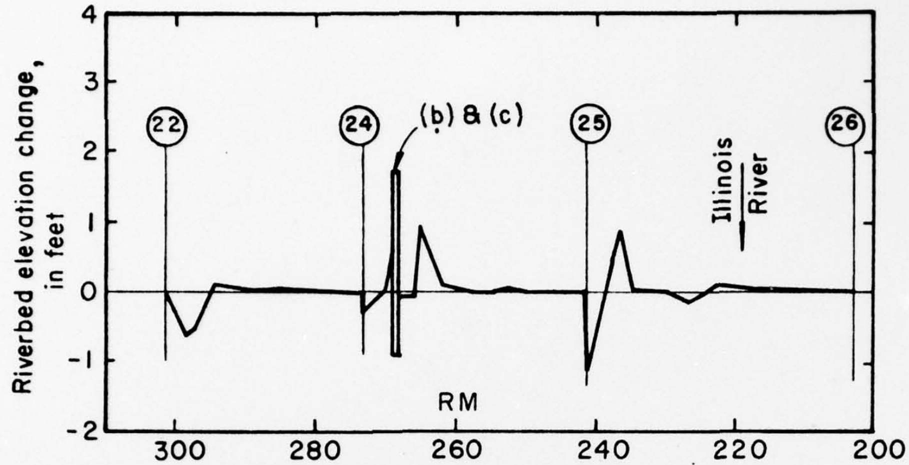
(a) Location of Dredging and Disposal Operations and General Riverbed Profile.



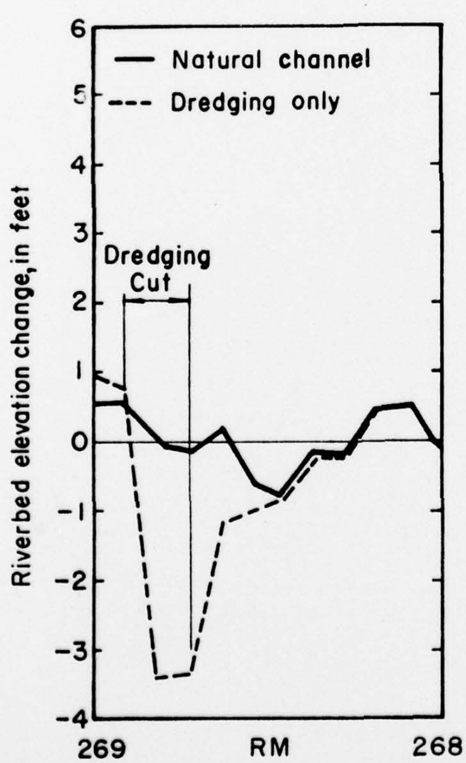
(b) Dispose All on Floodplain.

(c) Dispose 1/2 on Floodplain and 1/2 in Pool.

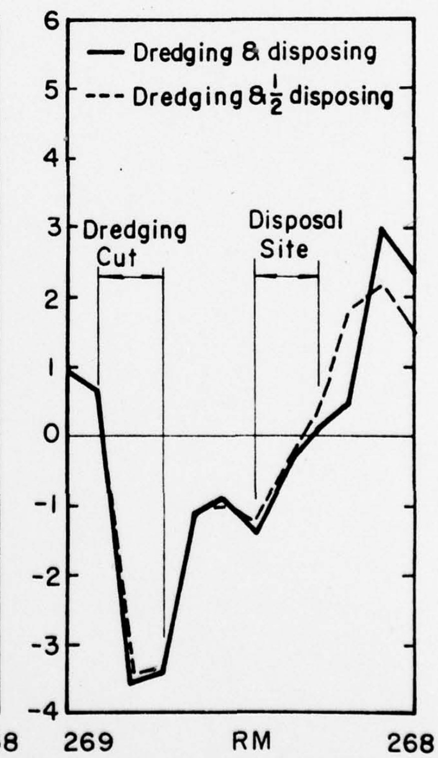
FIGURE 117. Average Riverbed Elevation Changes One Year after Dredging with Alternate Disposal Options (2-Year Annual Hydrograph) (after Simons et al., 1975).



(a) Location of Dredging and Disposal Operations and General Riverbed Profile.



(b) Dispose All on Floodplain.



(c) Dispose 1/2 on Floodplain and 1/2 in Pool.

FIGURE 118. Average Riverbed Elevation Changes One Year after Dredging with Alternate Disposal Options and a Small Annual Hydrograph (after Simons, et al., 1975).