THE RIVER ENVIRONMENT

A REFERENCE DOCUMENT

Prepared for

United States Department of the Interior
Fish and Wildlife Service
Twin Cities, Minnesota

Prepared by

Civil Engineering Department
Engineering Research Center
Colorado State University
Fort Collins, Colorado

D. B. Simons
P. F. Lagasse
Y. H. Chen
S. A. Schumm

December 1975

CER 75-76 DBS-PFL-YHC-SAS-14

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FOREWORD

This reference document was performed under Contract No. 14-16-0003-30,617 titled "A Study of the Geomorphology of the Upper Mississippi River," between the U.S. Fish and Wildlife Service and Colorado State University.

The study was supervised by Mr. William E. Martin, Regional Supervisor, Division of Ecological Services, U.S. Fish and Wildlife Service. Dr. D. B. Simons of Colorado State University was the principal investigator. He was assisted by Dr. P. F. Lagasse, Dr. S. A. Schumm, Dr. Y. H. Chen, and Dr. M. A. Stevens. Special thanks is due Dr. Lagasse for his efforts in organizing the material included in the report. The period of agreement was from July 1, 1974 to January 31, 1976.

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LIST OF SYMBOLS

A	Cross-sectional area of flow
A_d	Volume of sediment deposited on the channel bed per unit length of channel
A_f	Surface area of floodplain
A_x^y	Departure from a prismatic channel, $(\partial A/\partial x)_y$
a	Acceleration
amsl	Above mean sea level
C	Concentration of material in suspension in the fluid
C	Chezy discharge coefficient
$C/\sqrt{g}$	Chezy dimensionless discharge coefficient, $C/\sqrt{g} = V/\sqrt{gRS}$
C_b	Sediment concentration at or near the riverbank
C_F	Concentration of fine sediment (wash load)
C_o	Chezy discharge coefficient in steady uniform flow
C_s	Concentration of sediment discharge
C_T	Concentration of bed-material discharge
c	Subscript for critical conditions
cfs	Cubic feet per second, a unit of flow discharge
cu yd, cy	Cubic yards, a unit of volume
D	Depth of flow
d, D	Diameter of sediment particles
d_m, D_m	Effective sediment diameter
d_{50}, D_{50}	Median diameter of sediment particles for which 50 percent are finer than
e	Energy per unit mass
F	Total external forces
F	Width to depth ratio
Fr	Froude number, $V/\sqrt{gy}$

F_s	Shear force
f	Darcy-Weisbach resistance coefficient
fps	Feet per second, a unit of velocity
ft	Feet, a unit of length
f_s	Seepage force
g	Acceleration of gravity
H	Specific head, $V^2/2g + y_o$
H_L	Head loss (total)
H_T	Total head
i	Subscript for inside or initial
i_b	Fraction of the bed load represented by a given grain size D
i_s	Fraction of the suspended load represented by a given grain size D
i_t	Fraction of the total bed-material load represented by a given grain size D
K	Arbitrary constant
k_s	Height of the roughness element
L	Length
lb	Pounds, a unit of weight
ℓ	Subscript for lateral flow
M	Momentum change
M	Percentage of silt-clay
m	Mass
m	Subscript for model
mg/l	Milligram per liter, a unit of concentration
mm	Millimeter, a unit of length
n	Manning's roughness coefficient
n	Coordinate normal to flow direction
n_o	Manning's roughness coefficient in steady uniform flow

P	Wetted perimeter
PCP	Primary control point for the pool operation
p	Fluid pressure
p	Subscript for prototype
p	Volume of sediment in a unit volume of bed layer
ppm	Parts per million
Q	Discharge
Q'_b	Water discharge determining bed-load discharge
Q_B	Total bed load
Q_s	Sediment discharge
Q_s	Suspended load
Q_T	Total bed-material load
Q_{ss}	Total suspended load
q	Discharge per unit width
q_b	Bed-load discharge per unit width
q_s	Lateral sediment inflow per unit length of channel
q_s	Suspended-load discharge per unit width
q_t	Total bed-material discharge per unit width
R	Hydraulic radius, A/P
Re	Reynolds number, Vy/ν
RI	Recurrence interval
RM	River miles in the Mississippi River above the mouth of the Ohio River
r	Radius
r	Cylindrical coordinate
r	Subscript for ratio
r_c	Radius, center of bend
r_i	Radius, inside of bend

r_o	Radius, outside of bend
S	Slope
Sb	Silt clay percentage in the banks
S_C	Shape factor for the cross section of a river
Sc	Silt clay percentage in the bed
S_c	Critical slope
S_f	Slope of energy grade line
S_o	Slope of the bed
S_p	Shape factor of the sediment particles
S_R	Shape factor for the reach of a river
S_s	Specific gravity of solids
S_w	Slope of the water surface
s	Coordinate in the direction of flow
s	Sinuosity
sec	Seconds, a unit of time
sq	Square
t	Time variable
V	Mean velocity in the vertical
V	Mean velocity, Q/A
V_c	Critical velocity
$\forall$	Volume
V_*	Shear velocity, $V_* = \sqrt{gRS}$
v	Velocity at a point
W	Width of stream
W	Work done by the fluid
W_e	Weber number, $W_e = \dfrac{V}{\sqrt{\sigma/\rho L}}$
x	Einstein's multiplication factor in velocity distribution
x,y,z	Cartesian coordinate system

y	Depth
y_o	Normal depth of flow
y_o	Local depth of flow
y_c	Critical depth of flow
z	Rouse number, $z = \omega/\beta\kappa V_*$
z	Vertical distance
α	Abrasion coefficient
α	Kinematic energy coefficient
α	Swing angle of a channel
β	Coefficient relating diffusion coefficients
β	Momentum coefficient
β	Wear or sorting coefficient
γ	Specific weight of water-sediment mixture
γ_s	Specific weight of sediment (approximately 165 pounds per cubic foot)
γ_w	Specific weight of water (approximately 62.4 pounds per cubic foot)
Δ	Small increment
δ'	Laminar sublayer thickness, $\delta' = \frac{11.6\nu}{V_*}$
θ	Slope of angle of a channel
θ	Central angle
κ	von Karman universal velocity coefficient
μ	Dynamic viscosity
ν	Kinetic viscosity
ρ	Mass density of fluid
ρ_b	Bulk density of sediment forming the bed
ρ_s	Mass density of sediment
ρ_w	Mass density of water

Σ	Summation symbol
σ	Surface tension
τ	Shear stress
τ_c	Critical shear at which sediment motion is initialed
τ_o	Shear stress at the boundary
Φ_*	Dimensionless measure of bed-load transport defined by Einstei
ψ_*	Dimensionless measure of shear on a particle defined by Einste
ω	Fall velocity of sediment particles

Chapter 1

INTRODUCTION

1.1 Objectives

The purpose of The River Environment is to provide a reference document that presents the current state of knowledge on rivers and the river environment using fundamentals of fluid mechanics, geomorphology, hydraulics, and river mechanics. This handbook is intended to serve as a reference document, a teaching aid, and a guide for decision making relative to the evaluation of river response to alternate river development plans. As a basis for decision making this handbook is intended to provide a point of departure for the formulation and selection of river control and development alternatives considering all pertinent impacts of a geomorphic, hydrologic, and hydraulic nature. This basic reference document supplements a more detailed geomorphic study of the Upper Mississippi River and its major tributaries, as well as a detailed mathematical model study of selected portions of the Upper Mississippi River, and a study of response to riverine dredging operations and construction of river training works.

1.2 The Approach

The River Environment is organized to develop, first, a basic understanding of river morphology and river mechanics, and then, to expand on this basic knowledge and illustrate its application through an examination of the response of the Upper Mississippi River to the activities of man. Two areas of particular environmental concern, river response to channel stabilization and river response to dredging and disposal operations, are examined in detail.

Chapter 1 establishes the central theme of the reference document, that is, the absolute necessity of viewing rivers as dynamic physical/ ecological systems, and outlines ecological considerations and engineering requirements for river development programs. Research and training needs in the field of river mechanics also are highlighted. The basic concepts of river mechanics and river morphology are presented in Chapters 2 and 3, respectively. Chapter 4 provides the necessary baseline for application of geomorphic and hydraulic concepts of river mechanics to the Upper Mississippi River system by reviewing briefly the recent geologic history of the Upper Mississippi basin and establishing the morphology of the natural river prior to man's intervention. In Chapter 5, basic concepts are applied to the general analysis of river response problems and to the analysis of river response to man's intervention on a specific reach of the Upper Mississippi River. A detailed examination of river response to channel stabilization and river response to dredging in Chapters 6 and 7 provides the opportunity to expand on basic concepts, and, concurrently, affords insights into two aspects of man's development which are of primary environmental concern. Chapter 8 summarizes data necessary to an analysis of the response of river systems to development, and tabulates primary data sources. A detailed checklist of data needs is provided to serve as both a guide for data gathering and as an outline of basic considerations for the analysis of the impact of historical and proposed development activities on the river environment.

1.3 Rivers as Dynamic Systems

Frequently, environmentalists, river engineers, and others involved in river development, navigation, and flood control consider a river to be static; that is, unchanging in shape, dimensions, and pattern. However, an alluvial river generally is continually changing its position and shape as a consequence of hydraulic forces acting on its bed and banks and related biological forces interacting with these physical forces. These changes may be slow or rapid and may result from natural environmental changes or from changes caused by man's activities. When a river channel is modified locally, the change frequently causes modification of channel characteristics both up and down the stream. The response of a river to man-induced changes often occurs in spite of attempts to keep the anticipated response under control.

It should be clearly understood that a river is dynamic, that man-induced change frequently sets in motion a response that can be propagated for long distances, and that, in spite of their complexity, all alluvial rivers are governed by the same basic forces. Successful river development requires an understanding of these natural forces. It is absolutely necessary that river system design be based on competent knowledge of: (1) geologic factors, including soil conditions; (2) hydrologic factors, including possible changes in flows, runoff, and the hydrologic effects of changes in land use; (3) geometric characteristics of the stream, including the probable geometric alterations that will be activated by the changes that development will impose on the channel; (4) hydraulic characteristics such as depth, slope, and velocity of streams and the changes that may be expected in these characteristics in space and time; and (5) ecological/biological changes that will result from physical change and in turn will induce or modify physical changes.

1.3.1 Historical Evidence of the Natural Instability of Fluvial Systems

To emphasize the inherent dynamic qualities of river channels, evidence is cited below to demonstrate that most alluvial rivers are not static in their natural state. Indeed, scientists concerned with the history of landforms (geomorphologists), vegetation (plant ecologists), and the past activities of man (archaeologists), rarely consider the landscape as unchanging. Rivers, glaciers, sand dunes, and seacoasts are highly susceptible to change with time. Over a relatively short period of time, perhaps in some cases as long as man's lifetime, components of the landscape may seem to be relatively stable. Nevertheless stability cannot be automatically assumed. Rivers are, in fact, the most actively changing of all geomorphic forms.

Evidence from several sources demonstrates that river channels are continually undergoing changes of position, shape, dimensions, and pattern. In Figure 1-1, a section of the Mississippi River near Commerce

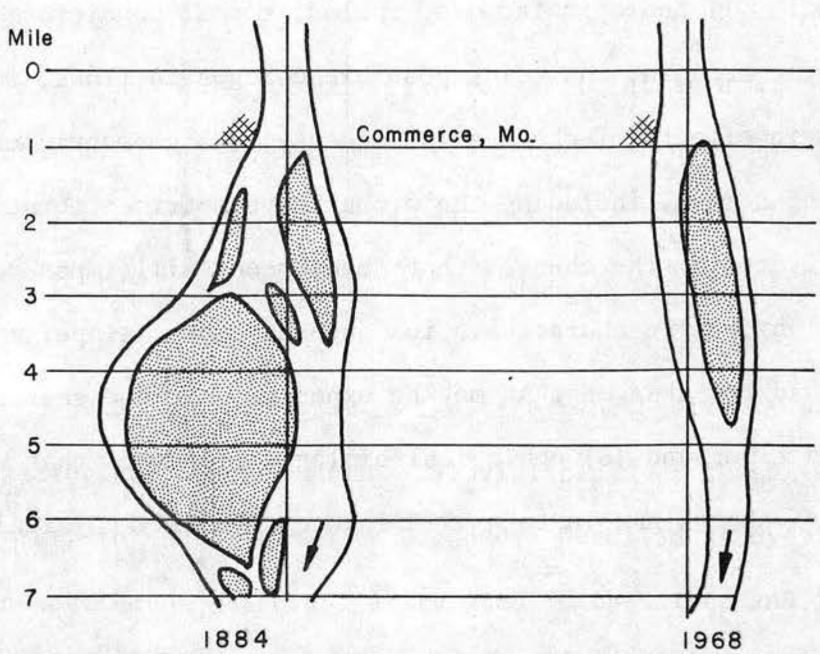


Figure 1-1 Comparison of the 1884 and 1968 Mississippi River Channel near Commerce, Missouri.

Missouri as it appeared in 1884 is compared with the same section as observed in 1968. In the lower 6 miles of this reach, the surface area has been reduced approximately 50 percent during this 84-year period. Some of this change has been natural and some has been the consequence of river development work.

In alluvial river systems, it is the rule rather than the exception that banks will erode, sediments will be deposited and floodplains, islands, and side channels will undergo modification with time. Changes may be very slow or dramatically rapid. Fisk's (1944) report on the Mississippi River and his maps (Figure 4-6) showing river position through time are sufficient to convince anyone of the innate instability of the Lower Mississippi River.

The Mississippi is our largest and most impressive river and because of its dimensions it has sometimes been considered unique. This is, of course, not so. Hydraulic and geomorphic laws apply at all scales of comparable landform evolution. The Mississippi may be thought of as a prototype of many rivers or as a much larger than prototype model of many alluvial sandbed rivers.

Rivers change position and morphology (dimensions, shape, pattern) as a result of changes of hydrology. Hydrologic characteristics can change as a result of climatic change over long periods of time, or as a result of natural stochastic climatic fluctuations (droughts, floods), or by man's modification of the hydrologic regime. For example, the major climatic changes of recent geological time (the last few million years of earth history) have triggered dramatic changes in runoff and sediment loads with corresponding channel alteration. As a result, major river changes of different types have occurred, including deep incision and deposition as sea level fluctuated, changes of channel geometry as a result of climatic and hydrologic changes, and obliteration or displacement of existing channels by continental glaciation. Climatic change and sea level change are interesting from an academic point of view but are not generally considered a direct cause of modern river instability.

Movement of the earth's crust is considered to be an important geologic agent causing modern river instability. The earth's surface in many parts of the world is undergoing measurable change by upwarping, subsidence or lateral displacement. As a result, the study of these changes (neotectonics) has become a field of major interest for many geologists and geophysicists. Such gradual surface changes can affect stream channels dramatically. For example, Wallace (1967) has shown that many small streams are clearly offset laterally along the San Andreas fault in California. Progressive lateral movement of this fault on the order of an inch per year has been measured. The rates of movement of faults are highly variable but an average rate of mountain building has been estimated by Schumm (1963) to be on the order of 25 feet per 1000 years. Seemingly insignificant in human terms, this rate is actually 0.3 inch per year or 3 inches per decade. For many river systems, a change of slope of 3 inches would be significant. (The slope of the energy gradient on the Lower Mississippi River is about 3 to 6 inches per mile).

An example of the cumulative effect of uplift is provided by archaeological studies on the alluvial plain of the Diyala River, a majo tributary to the Tigris near Baghdad. Detailed study of 6000 years of human settlement patterns, as related to the river and irrigation systems, shows an expansion of agricultural areas and then a progressive contraction. This is attributed to uplift, which steepened the gradient of the Diyala and caused it to incise to a depth of 15 meters, thereby leaving the irrigation canals high and dry. Although these changes occurred over perhaps 1000 years, there was an attempt to armor or pave the channel of the Diyala during a period of critical instability, when the users of water from the Diyala attempted to prevent further incision.

Of course, the geologist is not surprised to see drainage patterns that have been disrupted by uplift or some complex warping of the earth's surface. In fact, complete reversals of drainage lines have been documented. In addition, convexities in the longitudinal profile of both rivers and river terraces (these profiles are concave upward under normal development) have been detected and attributed to upwarping. Further, the progressive shifting of a river toward one side of its valley has resulted from lateral tilting. Major shifts in position of the Brahmaputra River toward the west are attributed by Coleman (1969) to tectonic movements. Hence, neotectonics should not be ignored as a possible cause of local river instability.

It is clear that rivers can display a remarkable propensity for change of position and morphology in time periods of a century. Hence rivers from the geomorphic point of view are unquestionably dynamic, but does this apply to modern rivers? It is probable that, during a period of several years, neither neotectonics nor a progressive climate change will have a detectable influence on river character and behavior. What then causes a river to appear relatively unstable from the point of view of the river engineer or the environmentalist? This instability is a result of the slow but implacable shift of a river channel through erosion and deposition at bends, the shift of a channel to form chutes and islands, and such activity as the cutoff of meander bends to form oxbow lakes.

Lateral migration rates are highly variable; that is, a river may maintain a stable position for long periods and then experience rapid movement. Much, therefore, depends on flood events, bank stability, permanence of vegetation on banks, and floodplain land use. A compilation of data by Leopold and Wolman (1957) shows that rates of lateral migration for the Kosi River of India approach 2500 feet per year. Rates of lateral migration for two major rivers in the United States are less dramatic, for example--Colorado River near Needles, California, 10 to 15 feet per year; Mississippi River near Rosedale, Mississippi, 158 to 630 feet per year.

Archaeologists also have provided clear evidence of natural channel changes in river morphology. For example, the number of archaeologic sites on floodplains decreases significantly with age simply because, as floodplains are modified by river migration, the earliest sites have been destroyed. Using such evidence, Lathrop (1968) working on the Rio Ucayali in the Amazon headwaters of Peru estimates that on the average a meander loop begins to form and cuts off in 5000 years. These loops have an amplitude of 2 to 6 miles and an average rate of meander growth of approximately 40 feet per year.

A study by Schmudde (1963) shows that about one-third of the floodplain of the Missouri River over the 170-mile reach between Glasgow and St. Charles, Missouri, was reworked by the river between 1879 and 1930. On the Lower Mississippi River, bend migration was on the order of 2 feet per year, whereas in the central and upper parts of the river below Cairo it was at times 1000 feet per year (Kolb, 1963). On the other hand, a meander loop pattern of the lower Ohio River has altered very little during the past thousand years (Alexander and Nunnally, 1972).

Although the dynamic behavior of perennial streams is impressive, the modification of rivers in arid and semiarid regions and especially of ephemeral (intermittent flow) stream channels is startling. A study of floodplain vegetation and the distribution of trees in different age groups led Everitt (1968) to the conclusion that about half of the Little Missouri River floodplain in western North Dakota was reworked in 69 years.

Historical and field studies by Smith (1940) show that floodplain destruction occurred during major floods on rivers of the Great Plains. An exceptional example of this is the Cimarron River of southwestern Kansas, which was 50 feet wide during the latter part of the 19th and first part of the 20th centuries (Schumm and Lichty, 1963). Following a series of major floods during the 1930's it widened to 1200 feet, and the channel occupied essentially the entire valley floor. During the decade of the 1940's a new floodplain was constructed, and the river width in 1960 was reduced to about 500 feet. Equally dramatic changes of channel dimensions have occurred along the North and South Platte Rivers in Nebraska and Colorado as a result of man's control of flood peaks by reservoir construction. Natural changes of this magnitude, due to changes in flood peaks, are exceptional, but they emphasize the dynamic nature and mobility of rivers.

In summary, archaeological, botanical, geologic, and geomorphic evidence supports the conclusion that most rivers are subject to constant change as a normal part of their morphologic evolution. Stable or static channels are the exception in nature.

1.3.2 A Survey of River Morphology and River Response

In the previous section it was established that rivers are dynamic and respond to changing environmental conditions. The direction and ext of the change depends on the forces acting on the system. The mechanics of flow in rivers is a complex subject that requires special study which is, unfortunately, not included in basic courses of fluid mechanics. Th major complicating factors in river mechanics are: (a) the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a river system and (b) the continual evolution of river channel patterns, channel geometry, bars, and forms of bed roughness with changing water and sediment discharge. To provide a glimpse of the factors controlling the response of a river to the actions of man and nature, a few simple hydraulic and geomorphic concepts are presented here. These concepts are treated in more detail in subsequent chapters.

Rivers are broadly classified as straight, meandering or braided or some combination of these classifications, but any changes that are imposed on a river may change its form. The dependence of river form on the slope of the channel bed, which may be imposed independent of the other river characteristics, is illustrated schematically in Figure 1-2. By changing the slope, it is possible to change the river from a meandering character that is relatively tranquil and easy to control to a braided river that varies rapidly with time, has high velocities, is subdivided by sandbars and carries relatively large quantities of sediment. Such a change could be caused by a natural or artificial cutoff. Conversely, it is possible that a slight decrease in slope could change an unstable braided river into a more stable meandering one.

Important changes in river morphology are caused by modification of discharge and sediment load. One of the first to consider this importan

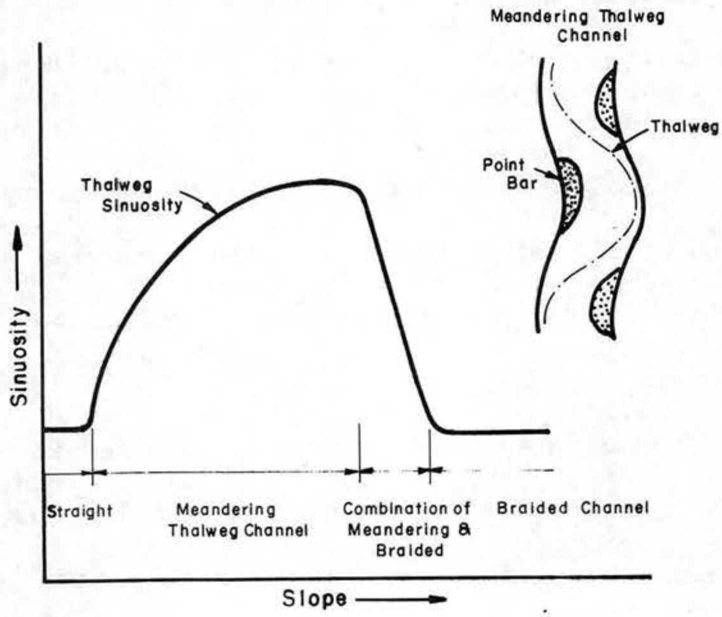


Figure 1-2 Sinuosity versus Slope with Constant Discharge.

problem was Lane (1955) who presented the relationship

$$QS \propto Q_s D_{50} \tag{1.1}$$

in which Q is the water discharge, S is the slope of the channel bed, Q_s is the bed-material discharge and D_{50} is a measure of the size of the channel bed material. This relation is both simple and useful. For example, if a dam is constructed across a river and traps bed-material sediments moving through the system, clear water is discharged immediately downstream, indicating a decrease in Q_s. Looking at Lane's relation for the river downstream of the dam the S must reduce on the left side of the equation to balance the decrease in Q_s on the right side, assuming constant Q and D_{50}. This implies degradation downstream of the structure. Lane's relation is very useful for qualitative analysis of stream response to both climatic and man-induced changes.

The significantly different channel dimensions, shapes, and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel morphology can be anticipated. Further, if changes in sinuosity and meander wavelength or in width and depth are required to compensate for a hydrologic change, then a long period of channel instability can be envisioned with considerable bank erosion and lateral shifting of the channel before stability is restored.

Changes in sediment and water discharge at a particular point or reach in a stream may have an effect ranging from some distance upstream to a point downstream where the hydraulic and geomorphic conditions can absorb the change. Thus, it is necessary to consider a channel reach as part of a complete drainage system. Artificial controls that could benefit the reach may, in fact, cause problems in the system as a whole. For example, flood control structures can cause downstream flood damage to be greater at reduced flows if the average hydrologic regime is changed so that the channel dimensions are actually reduced. Also, where major tributaries exert a significant influence on the main channel by introduction of large quantities of sediment, upstream control on the main channel may allow the tributary to intermittently dominate the system with deleterious results. If discharges in the main channel are reduced, sediments from the tributary that previously were eroded will no longer be carried away and serious aggradation with accompanying flood and navigation problems may arise.

An insight into the direction of change, the magnitude of change, and the time involved to reach a new equilibrium can be gained by studying the river in a natural condition, having knowledge of the sediment and water discharge, being able to predict the effects and magnitude of man's future activities, and applying to these a knowledge of geology, soils, hydrology, and hydraulics of alluvial rivers.

The current interest in ecology and the environment has resulted in increased awareness of the many problems that mankind can cause. Previous to the current concern with environmental impacts, very few people interested in rivers ever considered the long-term changes that were possible. It is imperative that anyone working with rivers, either with localized areas or entire systems, have an understanding of the many factors involved, and of the potential for change existing in the river system.

Methods of predicting river response include the use of physical and mathematical models. Engineers have long used small scale hydraulic models to assist them in anticipating the effect of altering conditions in a stretch of river . With proper awareness of the scale effects that exist, the results of hydraulic model testing can be extremely useful for this purpose. A more recent and perhaps more elegant method of predicting short- and long-term changes in rivers involves the use of mathematical models. To study a transient phenomena in natural alluvial channels, the equations of motion and continuity for sediment-laden water and the continuity equation for sediment can be used. These equations are powerful analytical tools for the study of unsteady flow problems. However, because of mathematical difficulties, practical solutions are usually obtained by numerical analysis using iteration procedures and digital computers. The increasing use of numerical mathematical models for flood and sediment routing, degradation, and aggradation studies and long-term channel development studies is indicative of their potential for contributing to the complex problems of river system development and response.

1.4 Environmental Considerations

Preserving environmental quality is of national interest and priority. The protection of fish and wildlife habitat and enhancement of related recreational benefits is basic to the national environmental protection movement. Plans to locate, design, construct, and maintain river modification projects must consider the social and ecological conditions that may have a bearing on the success of the planned project or could be affected adversely by the project. To do this, agencies involved with river modification activities must be able to recognize potential problems and conflicts.

1.4.1 Methods

River modification decisions should be made only after the potentia environmental impacts have been completely displayed and evaluated. This means that all feasible, alternative methods of doing the job and the environmental impacts of each must be understood and compared. Accordingly, an inventory of existing environmental resources (social, economic, and natural) is mandatory. Among other things, the inventory should show areas of unique vegetation, wildlife and aquatic life, endangered or threatened species, etc. In order to place these factors in proper perspective, ecologists must be consulted to spotlight the interdependence and interrelationship between the life forms and to predict ecological response to stresses caused by the various alternative plans. In addition to these necessary ecological studies, other interdisciplinary studies are required to understand the social and economic impact upon the environment of the various project alternatives.

Once the impacts are understood for each of the project alternatives, cost estimates for the viable alternatives can be developed to include enhancement, mitigation, and compensation features that should be built into the project proposal. Finally, a preferred alternative is selected that will allow the project to proceed with the least damage and greatest benefit to the total environment.

1.4.2 Identification of Existing Ecosystems

Pre-project consideration of the total ecosystem is an integral part of the plan. Ecological studies are necessary to document the present characteristics of the environment, to estimate the effects of development on the environment, and to provide the basis for selecting enhancement measures for minimizing any projected adverse effects.

The measures taken to assure that ecological studies are adequate include:

(1) Identify indicator biological species.

(2) Formulate studies for specific ecosystems that have high potential of being damaged or that may be used as a standard of environmental well-being.

(3) Combine and coordinate ecological studies with those developed for evaluating engineering needs.

1.4.3 Construction Effects

Although construction and maintenance activities may be of short duration as compared to the operating life of the project, these operations can have both immediate and long-term damaging effects on natural systems.

Erosion and other pollution control measures, impacts on area water quality and supply, and restoration of natural systems must be considered. The effects of construction and maintenance operations on navigation, the biota, water quality, aesthetics, recreation, water supply, flood damage prevention, ecosystems and, in general, the needs and welfare of the people require careful attention.

1.5 Engineering Requirements

Effects of river development, flood control measures and channel structures built during the last century, have proven the need for considering both delayed and far-reaching effects of any alteration man makes in a natural alluvial river system.

Because of the complexity of the processes occurring in natural flows involving the erosion and deposition of material, an analytical approach to the problem is difficult and time consuming. Most relationships describing river process have been derived empirically. Nevertheless, if a greater understanding of the principles governing the processes of river formation is to be developed, the empirically derived relations must be put in the proper context. Only in this way can the distinct limitations of the empirical relations be removed.

Mankind's attempts at controlling large rivers has often led to the situation described by J. H. Mackin when he wrote:

> "the engineer who alters natural equilibrium relations by diversion or damming or channel improvement measures will often find that he has the bull by the tail and is unable to let go--as he continues to correct or suppress undesirable phases of the chain reaction of the stream to the initial 'stress' he will necessarily place increasing emphasis on study of the genetic aspects of the equilibrium in order that he may work *with* rivers, rather than merely *on* them."

Through such experiences, man realizes that, to prevent or reduce the detrimental effects of any modification of the natural processes and state of equilibrium on a river, he must gain an understanding of the governing physical laws and become cognizant of the far-reaching effects of any attempt to control or modify a river's course.

1.5.1 Variables Affecting River Behavior

Variables affecting alluvial river channels are numerous and interrelated. Their nature is such that, unlike rigid boundary hydraulic problems, it is not possible to isolate and study the role of any individual variable.

Major factors affecting alluvial stream channel forms are:

(1) Stream discharge,

(2) Sediment load,

(3) Longitudinal slope,

(4) Bank and bed resistance to flow,

(5) Vegetation,

(6) Geology including types of sediments,

(7) Works of man.

The fluvial processes involved are extremely complicated and the variables of importance are difficult to isolate. Many laboratory and field studies have been carried out in an attempt to establish relationships among these and other variables. The problem has been more amenable to an empirical solution than an analytical one.

In an analysis of flow in alluvial rivers, the flow field is complicated by the constantly changing discharge. Significant variables are, therefore, quite difficult to relate mathematically. It is normal to list measurable or computable variables which effectively describe the processes occurring and then to reduce the list by making simplifying assumptions and examining relative magnitudes of variables, striving toward an acceptable balance between accuracy and limitations of obtaini data. When this is done, the basic equations of fluid motion may be sim plified (on the basis of valid assumptions) to describe the physical system.

It is the role of the succeeding chapters to present these variable define them, show how they interrelate, quantify their interrelations where feasible, and show how they can be applied to an analysis of river response to both historical and proposed development activities.

1.5.2 Basic Knowledge Required

In order to cope successfully with river engineering problems it is necessary to have specific background in: hydrology, hydraulics, erosio and sedimentation, river mechanics, soil mechanics, structures, economic ecology, and related subjects. As the public demands more comprehensive treatment of river development problems, the engineer must solicit the cooperative efforts of the hydrologist, geologist, geomorphologist, meteorologist, mathematician, statistician, computer programmer, systems engineer, soil physicist, soil chemist, biologist, ecologist, and economist. Professional organizations involving these talents should be encouraged to work cooperatively to achieve the long range research need and goals relative to river development and application of knowledge on national and international basis. Through an appropriate exchange of information between scientists working in these fields, opportunities fo success in all aspects of river development should be greatly enhanced.

1.5.3 Data Requirements

Large amounts of data pertaining to understanding the behavior of rivers have been acquired over a long period of time. Nevertheless, dat collection efforts are sporadic. Agencies should take a careful look at present data requirements needed to solve practical problems along with existing data. The basic type of information that is required includes: water discharge as a function of time, sediment discharge as a function of time, the characteristics of the sediments being transported by streams, the characteristics of the channels in which the water and sediment are transported, and the characteristics of watersheds and how they deliver water and sediment to the stream systems. Ecological data also is needed so that proper assessment can be made of the impact of river development upon the natural environment and vice versa. The problem of data requirements is of sufficient importance that it is treated in greater detail in subsequent chapters.

1.6 Research and Training Needs

When considering the future, it is essential to recognize the present state of knowledge pertaining to river hydraulics and then identify inadequacies in existing theories and encourage further research to help correct these deficits of knowledge. To correct such deficits there is need to take a careful look at existing data pertaining to rivers, future data requirements, research needs, training programs and methods of developing staff that can apply this knowledge to the solution of practical problems.

1.6.1 Adequacy of Current Knowledge

The basic principles of fluid mechanics involving application of continuity, momentum and energy concepts are well known and can be effectively applied to a wide variety of river problems. Considerable work has been done on the hydraulics of rigid boundary open channels and excellent results can be expected. The steady-state sediment transport of nearly uniform sizes of sediment in alluvial channels is well understood. There is good understanding of stable channel theory in noncohesive materials of all sizes. The theory is adequate to support design of stable systems in alluvial material or if necessary, designs can be made for appropriate types of stabilization treatments for canals and rivers to have them behave in a stable manner. A good understanding of plane-bed fluvial hydraulics exists. There have been extensive studies of the fall velocity of noncohesive sediments in static fluids to provide knowledge about the interaction between the particle and flui so essential to the development of sediment transport theories.

In conclusion, available concepts and theories which can be applied to the behavior of rivers are extensive. However in many instances, onl empirical relationships have been developed and these are pertinent to specific problems only. Consequently, a more basic theoretical understanding of flow in the river systems needs to be developed.

With respect to many aspects of river mechanics, it can be conclude that knowledge is available to cope with the majority of river problems. On the other hand, the number of individuals who are cognizant of existi theory and can apply it successfully to the solution of river problems i limited. Particularly, the number of individuals involved in the actual solution of applied river mechanics problems is very small. There is a specific reason for this deficit of trained personnel. Undergraduate engineering educators in the universities in the United States, and in the world for that matter, devote only a small amount of time to teachi hydrology, river mechanics, channel stabilization, fluvial geomorphology and related problems. It is not possible to obtain adequate training in these important topics except at the graduate level and only a limite number of universities and institutions offer the required training in these subject areas. There is a great need at this time to train people to cope with river problems.

1.6.2 Research Needs

As knowledge of river hydraulics is reviewed, it becomes quite obvious that many things are not adequately known. Research needs relative to river morphology and hydraulics are particularly urgent and promise a rather quick return. The classifications of rivers is one such area. Different kinds of rivers should be studied separately because the factors governing their behavior may not be the same. Stabilization of rivers and bank stability of river systems needs further consideration. Also, the study of bed forms generated by the interaction between the water and sediment in the river system deserves further study. The types of bed forms have been identified but theories pertaining to their development are inadequate. Simple terms have been used to describe the characteristics of alluvial material of both cohesive and noncohesive types; a comprehensive look at the characteristics of materials is warranted.

Other important research problems include the fluid mechanics of the motion of particles, secondary currents, three-dimensional velocity distributions, fall velocity of particles in turbulent flow and the application of remote sensing techniques to hydrology and river mechanics. The physical modeling of rivers followed by prototype verification, mathematical modeling of river response followed by field verification, mathematical modeling of water and sediment yield from small watersheds and studies of unsteady sediment transport are areas in which significant advances can be made.

Finally, the results of these efforts must be presented in such a form that they can be easily taught and put to practical use.

1.6.3 Training

It has been pointed out that engineering training is somewhat inad quate in relation to understanding the development of rivers. There i need to consider better ways to train individuals to disseminate existi knowledge in this important area. The training of individuals could be accomplished by conducting seminars, conferences or short courses in institutions in the spirit of continuing education. There should be an effort to improve the curriculum of university education made available, particularly at the undergraduate level. At the very minimum such curriculum should strive to introduce concepts of fluvial geomorphology, river hydraulics, erosion and sedimentation, environment considerations and related topics.

To overcome the deficit of knowledge, manuals, handbooks and reference documents should be prepared. Publication of material pertaining to rivers should be encouraged. This material can be and is being published to some degree in the proceedings of conferences, in journals, and in textbooks. Better use of informative fil could be made. Similarly, television and video tapes can be economical prepared and utilized in instructional situations. Television cameras are available that enable the teacher to record and take field situatio directly into the classroom for class consideration.

Formal classroom training should be supported with field trips and laboratory demonstrations. Laboratory demonstrations are an inexpensiv method of quickly and effectively teaching the fundamentals of river mechanics and illustrating the behavior of structures. These demonstra tions should be followed by field trips to illustrate similarities and differences between phenomena in the laboratory and in the field.

Finally, larger numbers of disciplines should be involved in the training programs. Cooperative studies should involve research personnel, practicing engineers and people from the many different disciplines with an interest in rivers.

Chapter 2

RIVER MECHANICS

2.1 The Hydraulics of Open Channel Flow

2.1.1 Introduction

In this section the fundamentals of rigid boundary open channel flow are described. In open channel flow, the water surface is not confined and surface configuration, flow pattern and pressure distribution within the flow depend on gravity. In rigid boundary open channel flow, no deformations or movements of the bed and banks are considered. Mobile boundary hydraulics is discussed in subsequent sections. In this chapter, we restrict ourselves to one-dimensional analysis; that is, the direction of velocity and acceleration are large only in one direction and are so small as to be negligible in all other directions.

Open channel flow can be classified as: (1) uniform or nonuniform flow, (2) steady or unsteady flow, (3) laminar or turbulent flow, and (4) tranquil or rapid flow. In uniform flow, the depth and discharge remain constant with respect to space. Also the velocity at a given depth is the same everywhere. In steady flow, no change occurs with respect to time. In laminar flow, the flow field can be characterized by layers of fluid, one layer not mixing with adjacent ones. Turbulent flow on the other hand is characterized by random fluid motion. Tranquil flow is distinquished from rapid flow by a dimensionless number called the Froude number (Fr). If $Fr < 1$, the flow is tranquil; if $Fr > 1$, the flow is rapid, and if $Fr = 1$, the flow is called critical.

Open channel flow can be nonuniform, unsteady, turbulent and rapid at the same time. Because the classifying characteristics are independent, sixteen different types of flow can occur. These terms, uniform or nonuniform, steady or unsteady, laminar or turbulent, rapid or tranquil, and the two dimensionless numbers (the Froude number and Reynolds number) are more fully explained below.

2.1.2 Definitions

Velocity: The velocity of a fluid particle is the time rate of displacement of the particle from one point to another. Velocity is a vector quantity. That is, it has both magnitude and direction. The mathematical representation of the fluid velocity is

$$V = \frac{ds}{dt} \qquad (2.1$$

Streamline: An imaginary line within the flow which is everywhere tangent to the velocity vector is called a streamline.

Acceleration: Acceleration is the time rate of change of the velocity vector, either in magnitude or direction or both. Mathematically, acceleration is expressed by the total derivative of the velocit vector or

$$\frac{DV}{Dt} = \frac{dv_s}{dt} + \frac{dv_n}{dt} \qquad (2.2)$$

where the subscript s is along the streamline and n refers to the direction normal to the streamline. The tangential acceleration component is

$$a_s = \frac{dv_s}{dt} \qquad (2.3)$$

and the normal acceleration component is

$$a_n = \frac{dv_n}{dt} \qquad (2.4)$$

Uniform flow: In uniform flow, there is no change in velocity along a streamline with distance; that is,

$$\frac{\partial v_s}{\partial s} = 0 \qquad \text{and} \qquad \frac{\partial v_n}{\partial n} = 0$$

Nonuniform flow: In nonuniform flow, velocity varies with position so

$$\frac{\partial v_s}{\partial s} \neq 0 \qquad \text{and} \qquad \frac{\partial v_n}{\partial n} \neq 0$$

Examples of nonuniform flow include: flow around a bend ($\partial v_n/\partial n \neq 0$) and flow in expansions or contractions ($\partial v_s/\partial s \neq 0$).

Steady flow: In steady flow, the velocity at a point does not change with time; that is,

$$\frac{\partial v_s}{\partial t} = 0 \qquad \text{and} \qquad \frac{\partial v_n}{\partial t} = 0$$

Unsteady flow: In unsteady flow, the velocity at a point varies with time so

$$\frac{\partial v_s}{\partial t} \neq 0 \qquad \text{and} \qquad \frac{\partial v_n}{\partial t} \neq 0$$

Examples of unsteady flow are channel flows with waves, flood hydrographs, and surges. Unsteady flow is difficult to analyze unless the time changes are small.

Laminar flow: In laminar flow, the mixing of the fluid and momentum transfer is by molecular activity.

Turbulent flow: In turbulent flow the mixing of the fluid and momentum transfer is by random fluctuations of finite "lumps" of fluid.

The flow is laminar or turbulent depending on the value of the Reynolds number ($Re = \rho VL/\mu$), which is a dimensionless ratio of the inertial forces in the system to the viscous forces. Here ρ and μ

are the density and dynamic viscosity of the fluid, V is the fluid velocity and L is a characteristic dimension, usually the depth in open channel flow. In laminar flow, viscous forces are dominant and Re is relatively small. In turbulent flow, Re is large; that is, inertial forces are very much greater than viscous forces.

In turbulent flow over a hydraulically smooth boundary (see Section 2.1.4.2) viscous forces near the boundary are the dominant resistance to flow. With a hydraulically rough boundary form drag is more significant than viscous drag and is the dominant reason for resistance to flow. Between these two types of roughnesses there is an intermediate condition where viscosity and form drag both affect the flow.

Turbulent flows are predominant in nature. Laminar flow occurs only infrequently in open channel flow.

Tranquil flow: In open channel flow, the flow pattern, surface configuration, depth, and changes in these quantities in response to changes in channel geometry depend on the Froude number ($Fr = V/\sqrt{gL}$) which is the ratio of inertia forces in the system to gravitational forces. The Froude number is also the ratio of the flow velocity to the velocity of a small gravity wave in the flow. When $Fr < 1$, the flow is tranquil, and surface waves propagate upstream as well as downstream. Control of tranquil flow depth is always downstream.

Rapid flow: When $Fr > 1$, the flow is rapid and surface disturbances can propagate only in the downstream direction. Control of rapid flow depth is always at the upstream end of the rapid flow region. When Fr = 1.0, the flow is critical and surface disturbances remain stationary in the flow.

2.1.3 Basic Principles

The basic equations of flow in open channels are derived from the three conservation laws. These are: (1) the conservation of mass, (2) the conservation of linear momentum, and (3) the conservation of energy. The conservation of mass is another way of stating that (except for mass-energy interchange) matter can neither be created nor destroyed. The principle of conservation of linear momentum is based on Newton's second law of motion which states that a mass (of fluid) accelerates in the direction of and in proportion to the applied forces on the mass. The principle of conservation of energy is based on the First Law of Thermodynamics.

In the analysis of flow problems, much simplification can result if there is no acceleration of the flow or if the acceleration is primarily in one direction, the accelerations in other directions being negligible. However, a very inaccurate analysis may occur if one assumes accelerations are small or zero when in fact they are not. The developments in this reference document assume one-dimensional flow and the derivations of the equations utilize a control volume. A control volume is an isolated volume in the body of the fluid, through which mass, momentum, and energy can be convected. The control volume may be assumed fixed in space or moving with the fluid.

A stream tube is a control volume bounded by streamlines. Fluid containing mass, momentum and energy enters at 1 and leaves at 2, (see Figure 2-1). As steady flow is assumed there can be no storage within the stream tube. Acting on the boundaries are the fluid pressures p_1 and p_2 and shear stresses τ.

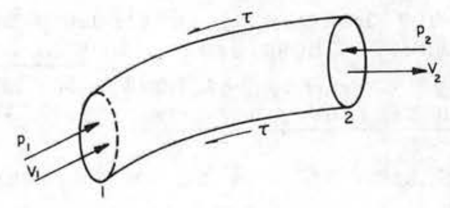


Figure 2-1 A Stream Tube as a Control Volume.

2.1.3.1 Conservation of Mass (Continuity Equation). The principle of conservation of mass is that matter can be neither created nor destroyed. Mathematically for the stream tube (assuming steady flow) in Figure 2-1, the principle can be expressed

$$\rho_1 A_1 V_1 = \rho_2 A_2 V_2 , \tag{2.5}$$

where ρ_1 is the mass density of the fluid entering the stream tube with velocity V_1 at Section 1 with cross section area A_1 and leaving with a mass density ρ_2 and velocity V_2 at Section 2 with area A_2. Note that the velocity vectors V_1 and V_2 are perpendicular to the areas A_1 and A_2. The weight rate of flow is written $\gamma_1 A_1 V_1 = \gamma_2 A_2 V_2$ where γ is the specific weight and is given by $\gamma = \rho g$, where g is the acceleration of gravity.

If the mass density is constant (incompressible flow) then Equation (2.5) becomes the Continuity Equation:

$$Q = A_1 V_1 = A_2 V_2 \tag{2.6}$$

where Q is the flow discharge. For a more rigorous development of the continuity equation see Appendix B.

2.1.3.2 Conservation of Linear Momentum (Momentum Equation). The conservation of momentum is based on Newton's second law of motion applied to the fluid within the control volume. The equation expressing Newton's second law of motion is:

$$F_i = ma_i \tag{2.7}$$

where the left side of the equation is the summation of all external forces acting on the fluid (pressure and shear forces on the boundary) and the right side of the equation includes the product of the mass and the change in velocity (acceleration) either in direction or magnitude that results from the force. It is a vector equation, hence, the subscript i denotes the components of an orthogonal coordinate system.

The momentum equation is derived rigorously (see Appendix B) by writing the Newtonian equation in the differential impulse momentum form

$$d\,F_i dt = d\,M_i \tag{2.8}$$

For steady flow with constant density, integration between sections one and two of Figure 2-1 yields

$$\Sigma F_i = M_{i2} - M_{i1} \tag{2.9}$$

where ΣF_i is the summation of all forces acting in the i direction on the fluid within the control volume; M_i is the change in momentum between Sections 1 and 2 resulting from the external forces (momentum flux into and out of the control volume represented by the stream tube in Figure 2-1)

$$M_i = \rho \int v_i dQ \tag{2.10}$$

For channel flows it is more convenient to treat average velocities at a cross section, using an integrated momentum flux and a correction coefficient β (see Appendix B) to account for the velocity variation in the cross section. M_i then is

$$M_i = \beta \rho Q V_i \tag{2.11}$$

With these changes Equation (2.9) for steady, incompressible flow becomes

$$\Sigma F_i = \rho Q[(\beta V_i)_2 - (\beta V_i)_1] \tag{2.12}$$

The coefficient β normally varies from 1 to 1.05 and usually may be assumed to be unity. In cartesian coordinates with directions x, y, and z, and for $\beta = 1$ Equation (2.12) becomes

$$\Sigma F_x = \rho Q(V_{x_2} - V_{x_1}) \tag{2.13}$$

$$\Sigma F_y = \rho Q(V_{y_2} - V_{y_1}) \tag{2.14}$$

$$\Sigma F_z = \rho Q(V_{z_2} - V_{z_1}) \tag{2.15}$$

2.1.3.3 Conservation of Energy (Energy Equation). Energy can exist in many forms (light, heat, chemical, mechanical, etc.) and every process of nature involves conversion of energy from one form to another. In hydraulics the principal forms of energy considered are

E_K, Kinetic energy per unit mass ($V^2/2$)

E_p, Pressure energy per unit mass (p/ρ)

E_e, Potential energy per unit mass (gz)

E_H, Heat energy per unit mass (gH_L)

E_M, Mechanical energy per unit mass added by a pump or subtracted by a turbine (gH_m)

The law of conservation of energy (First Law of Thermodynamics) states that energy can neither be created nor destroyed and the above forms then must sum to a constant. That is

$$E_K + E_p + E_e + E_H + E_m = \text{Constant} \tag{2.16}$$

or dividing each term by the local gravitational acceleration

$$V^2/2g + p/\gamma + z + H_L + H_m = \text{Constant} \tag{2.17}$$

In this form, the energy within a fluid system is based on unit weights of fluid. The energy equation is a scalar equation and accounts for net efflux of energy from a control volume plus input of energy from outside the control volume less the energy expended (transferred) from the control volume to its surroundings. As the present development is for a control volume which is a stream tube, and a stream tube becomes a streamline as its cross-sectional area is gradually reduced, Equations (2.16) and (2.17) apply to a streamline. It is convenient also to be able to apply the energy equation to a larger control volume, say one which encompasses an entire channel cross section and a measureable length of the channel. For this control volume, it is more convenient to use an average cross-sectional velocity. In general, the velocity differs for different streamlines in a cross section. Consequently a kinetic energy correction factor, α, (see Appendix B) must be introduced. Thus, Equation (2.17), neglecting mechanical energy input for open channel flow may be rewritten as

$$\alpha_1 \frac{V_1^2}{2g} + \frac{p_1}{\gamma} + z_1 = \alpha_2 \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + z_2 + H_L \tag{2.18}$$

The subscripts 1 and 2 refer to upstream and downstream cross sections of the channel respectively. When $\alpha = 1$ and $H_L = 0$, Equation (2.18) is commonly referred to as the Bernoulli equation which can also be derived from the equation of motion (F = ma). If derived from the equation of motion it is called the Euler equation.

<u>Assumptions and limitations of the energy equation</u>: The energy equation as expressed by Equation (2.18) is the energy change per unit weight of fluid, when it flows from one arbitrary cross section of the channel to another farther downstream. It was derived with the following assumptions and has the following limitations.

1. The flow is steady.

2. The flow is incompressible (Equation (2.18) can be used for compressible flow if care is used to take into consideration density changes and the gas laws).

3. The flow is irrotational. Turbulent flow of water in natural channels usually meets this requirement although flow around bends may be rotational (see Figure 2-2).

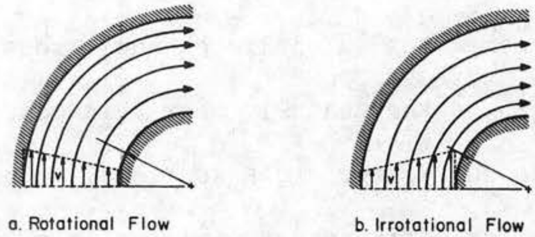


Figure 2-2 Velocity Distribution for Rotational and Irrotational Flow in a Bend.

4. The Kinetic energy coefficient α for flows in canals and natural channels varies from 1.0 for relatively uniform channels with

no flow separation and little or no velocity variations in the cross section to 1.15 for nonuniform channels with bends and large (200%) velocity variations in the cross section. However, if there are large overbank flow areas, large separation zones, and considerable divergence in the flow, α's as large as 3.0 have been computed. In cases such as these it is better to subdivide the cross section and apply the energy equation independently for each subdivision. For subdivided reaches, or fairly uniform channels, α is assumed unity because the additional refinement of applying an α of 1.05 is not needed.

5. The flow is assumed one-dimensional, that is, accelerations and resultant forces in the flow-wise direction are much greater than in the normal (perpendicular) directions.

6. The head loss H_L in Equation (2.18) is the amount of energy per unit weight of fluid that is converted to heat in the flow process. This is called friction heat and results from the shearing stresses both at the boundary and internally in the flow. The heat energy generated by the flow in an isolated system would increase the temperature of the flow; however, in most flow systems it is conducted to the boundaries and dissipated into the environment and lost from the flow. Entropy is the term used as a measure of the energy lost from the system or unavailable in the system to perform useful work.

7. The Energy equation does not indicate a maximum velocity nor a minimum pressure. However, a liquid will boil when the pressure is equal to its vapor pressure which limits the application of Equation (2.18).

2.1.3.4 Hydrostatics. When the only forces acting on the fluid are pressure and fluid weight and for steady uniform flow (or for zero flow), the energy equation reduces to the equation of hydrostatics

$$\frac{p}{\gamma} + z = \text{Constant} \tag{2.19}$$

However, when there is acceleration, the piezometric head varies, that is, the piezometric head $(P/\gamma + z)$ is not constant in the flow. This is illustrated in Figure 2-3. In Figure 2-3a the pressure at the bed is equal to γy_o whereas in the curvilinear flow (Figure 2-3b) the pressure is larger than γy_o because of the acceleration resulting from a change in direction.

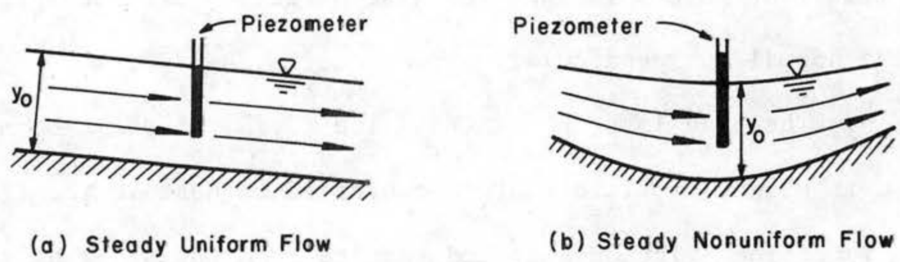


Figure 2-3 Pressure Distribution in Steady Uniform and in Steady Nonuniform Flow.

In general, when fluid acceleration is small (as in gradually varied flow) the pressure distribution is considered hydrostatic. However, for rapidly varying flow where the streamlines are converging, diverging or have substantial curvature (curvilinear flow), fluid accelerations are not small and the pressure distribution is not hydrostatic.

In Equation (2.19), the constant is equal to zero for gage pressure at the free surface of a liquid, and for flow with hydrostatic pressure throughout (steady, uniform flow or gradually varied flow) it follows that the pressure head p/γ is equal to the vertical distance below the free surface. In sloping channels with steady uniform flow, the pressure head p/γ at a depth y below the surface is equal to

$$\frac{p}{\gamma} = y \cos\theta \tag{2.20}$$

Note that y is the depth (perpendicular to the water surface) to the point, as shown in Figure 2-4. For most channels, θ is small and $\cos\theta \simeq 1$.

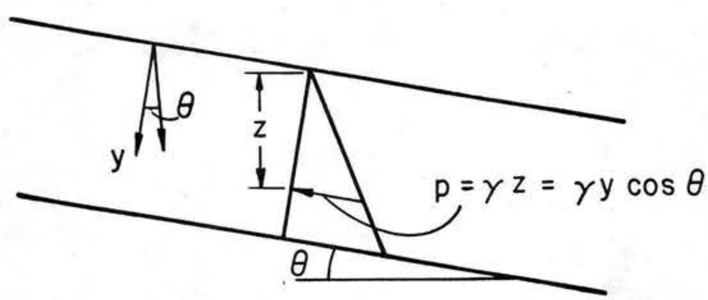


Figure 2-4 Pressure Distribution in Steady Uniform Flow on Large Slopes.

2.1.4 Steady Uniform Flow

2.1.4.1 Introduction. In steady, uniform open channel flow there are no accelerations, streamlines are straight and parallel and the pressure distribution is hydrostatic. The slope of the water surface S_w, bed surface S_o and the energy grade line S_f are equal to each other and equal to the head loss term in the energy equation divided by the length of reach (see Figure 2-5).

The energy equation from Section 1 to Section 2 is:

$$z_1 + y_1 \cos\theta + \alpha \frac{V_1^2}{2g} = z_2 + y_2 \cos\theta + \alpha \frac{V_2^2}{2g} + H_L \tag{2.21}$$

for most natural channels θ is small and $y \cos\theta \approx y$ and in general $\alpha \approx 1$. The slopes of the bed, water surface and energy grade line are

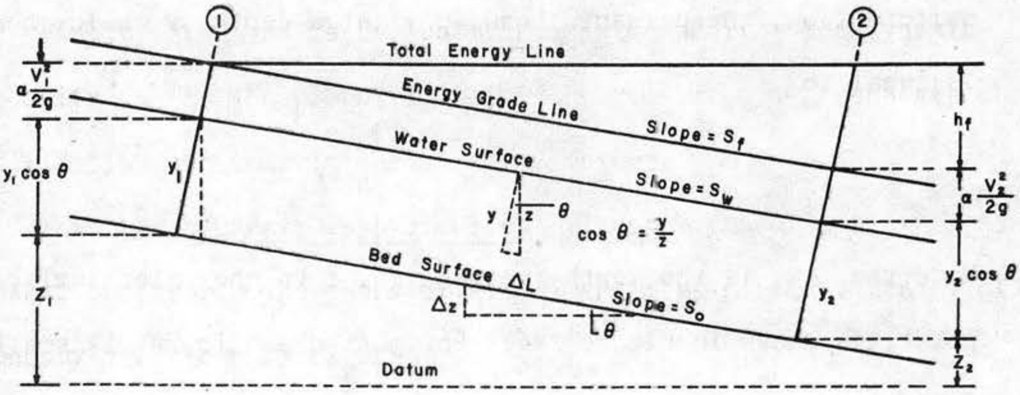


Figure 2-5 Definition Sketch for Steady Uniform Flow.

$$S_o = \sin\theta = \frac{z_1 - z_2}{L} \tag{2.22}$$

$$S_w = \frac{(z_1+y_1)-(z_2+y_2)}{L} = \frac{z_1 - z_2}{L} \tag{2.23}$$

$$S_f = \frac{H_L}{L} = \frac{z_1 - z_2}{L} \tag{2.24}$$

Steady uniform flow is an idealized concept for open channel flow and even in laboratory flumes is difficult to obtain. However, for many applications the flow is steady and the changes in width, depth or direction (resulting in nonuniform flow) are so small that the flow can be considered uniform. In other cases when changes occur over a long distance, the flow is considered gradually varied flow. However, in other cases the changes are large enough that the accelerations cannot be neglected (rapidly varied flow, and flow around bends). Steady, uniform flow is probably the simplest of the flow fields to analyze and provides a framework for analysis of other flow conditions.

Variables of interest for steady uniform flow are 1) the mean velocity V, 2) the discharge Q, 3) the velocity distribution $v(y)$ in the vertical, 4) the head loss H_L through the reach, and 5) the shear stress, both local τ and at the bed τ_o. The variables are interrelated and which variable we determine and how we determine it depends on the data available. For example, if the discharge and cross section area are known then the mean velocity is easily determined from continuity. However, if the discharge is not known then H_L must be determined from some suitable equation, and subsequently Mannings or Chezy equation is used to determine the velocity.

2.1.4.2 Shear-Stress and Velocity Distribution. Shear stress τ is the internal fluid stress which resists deformation. The shear stress exists only when fluids are in motion. It is a tangential stress in contrast to pressure which is a normal stress.

The local shear stress at the interface between the boundary and the fluid can be determined quite easily if the boundary is hydraulically smooth; that is, if the roughness at the boundary is submerged in a viscous sublayer as shown in Figure 2-6. Here, the thickness of the

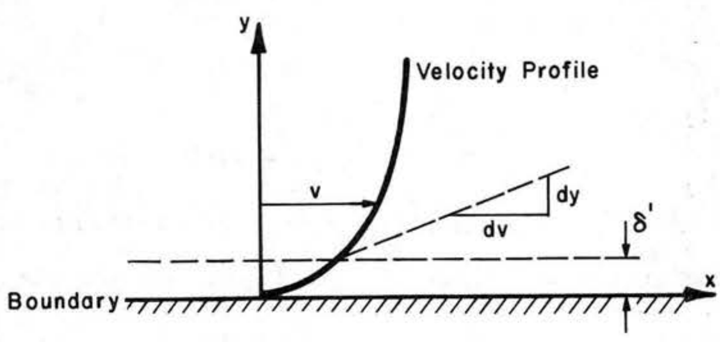


Figure 2-6 Hydraulically Smooth Boundary.

laminar sublayer is δ'. In laminar flow, the shear stress at the boundary is

$$\tau_o = \mu\left(\frac{dv}{dy}\right)_{y=0} \tag{2.25}$$

The velocity gradient is evaluated at the boundary. The dynamic viscosity μ is the proportionality constant relating boundary shear and velocity gradient in the viscous sublayer.

When the boundary is hydraulically rough, there is no viscous or laminar sublayer. The paths of fluid particles in the vicinity of the boundary are shown in Figure 2-7.

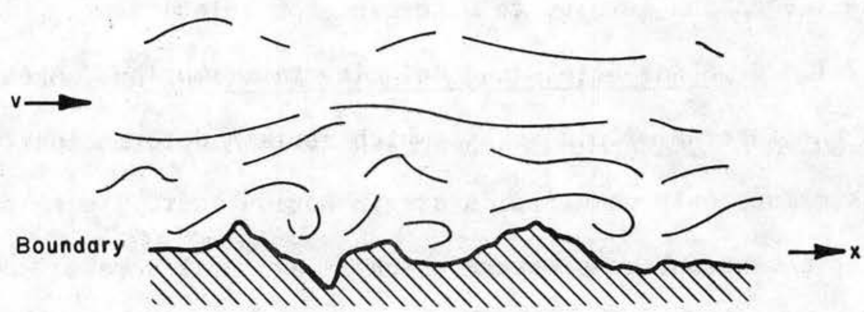


Figure 2-7 Hydraulically Rough Boundary.

The velocity at a point near the boundary fluctuates randomly about a mean value. The random fluctuation in velocity is called turbulence. For the hydraulically rough boundary,

$$\tau_o \neq \mu \frac{dv}{dy}$$

so another method of expressing τ_o is required. So far, the only satisfactory way of determining the boundary shear stress at a rough boundary has been experimentally.

Using different approaches Prandtl (1925) and von Kármán (1930) derived the same equation for turbulent boundary shear in steady uniform flow,

$$\tau = \rho \kappa^2 y^2 \left(\frac{dv}{dy}\right)^2 \tag{2.26}$$

Equation (2.26) can be rearranged to the form

$$\frac{dv}{dy} = \frac{\sqrt{\tau_o/\rho}}{\kappa y} \tag{2.27}$$

where κ is the von Kármán universal velocity coefficient. For rigid boundaries κ has the average value of 0.4. The term τ_o is the shear stress at the bed. The term $(\tau_o/\rho)^{1/2}$ has the dimensions of velocity and is called the shear velocity. Integration of (2.27) yields

$$\frac{v}{\sqrt{\tau_o/\rho}} = \frac{1}{\kappa} \ln \frac{y}{y'} = \frac{2.31}{\kappa} \log \frac{y}{y'} \tag{2.28}$$

Here ln is the logarithm to the base e and log is the logarithm to the base 10. The term y' results from evaluation of the constant of integration assuming $v = 0$ at some distance y' above the bed.

The term y' depends on the flow and has been experimentally determined. Many experiments have resulted in characterizing turbulent flow into three general types:

(1) Hydraulically smooth boundary turbulent flow where the velocity distribution, mean velocity, and resistance to flow are independent of the boundary roughness of the bed but depend on fluid viscosity. Then with δ' equal to $11.6\nu/\sqrt{\tau_o/\rho}$, $y' = \delta'/107$.

(2) Hydraulically rough boundary turbulent flow where velocity distribution, mean velocity and resistance to flow are independent of viscosity and depend entirely on the boundary roughness. For this case, $y' = k_s/30.2$ where k_s is the height of a roughness element.

(3) Transition where the velocity distribution, mean velocity and resistance to flow depends on both fluid viscosity and boundary roughness. Then

$$\frac{\delta'}{107} < y' < \frac{k_s}{30.2}$$

There are many forms of Equation (2.28) depending on the method of expressing y'. The Einstein method of expressing y' is well-known. The Einstein form of the Karman-Prandtl velocity distribution, mean velocity and resistance to flow equations are:

$$\frac{v}{V_*} = 5.75 \log \left(30.2 \frac{xy}{k_s}\right) = 2.5 \ln \left(30.2 \frac{xy}{k_s}\right) \tag{2.29}$$

$$\frac{V}{V_*} = \frac{C}{\sqrt{g}} = 5.75 \log \left(12.27 \frac{xy_o}{k_s}\right) = 2.5 \ln \left(12.27 \frac{xy_o}{k_s}\right) \tag{2.30}$$

where

x = a coefficient given in Figure 2-8

k_s = the height of the roughness elements. For sand channels, k_s is the D_{65} of the bed material

v = the local mean velocity at depth y

y_o = the depth of flow

V = the depth-averaged velocity

V_* = the shear velocity $\sqrt{\tau_o/\rho}$
$= \sqrt{gRS_f}$

τ_o = the shear stress at the boundary
$= \gamma RS_f$

R = the hydraulic radius equal to the flow area divided by the wetted perimeter

S_f = the slope of the energy grade line

δ' = the thickness of the viscous sublayer
$= \frac{11.6\nu}{V_*}$

C = the Chezy discharge coefficient

2.1.4.3 <u>Empirical Velocity Equations</u>. Because of the difficulties involved in determining the shear and hence the velocity distribution in turbulent flows an empirical approach to determine mean velocities

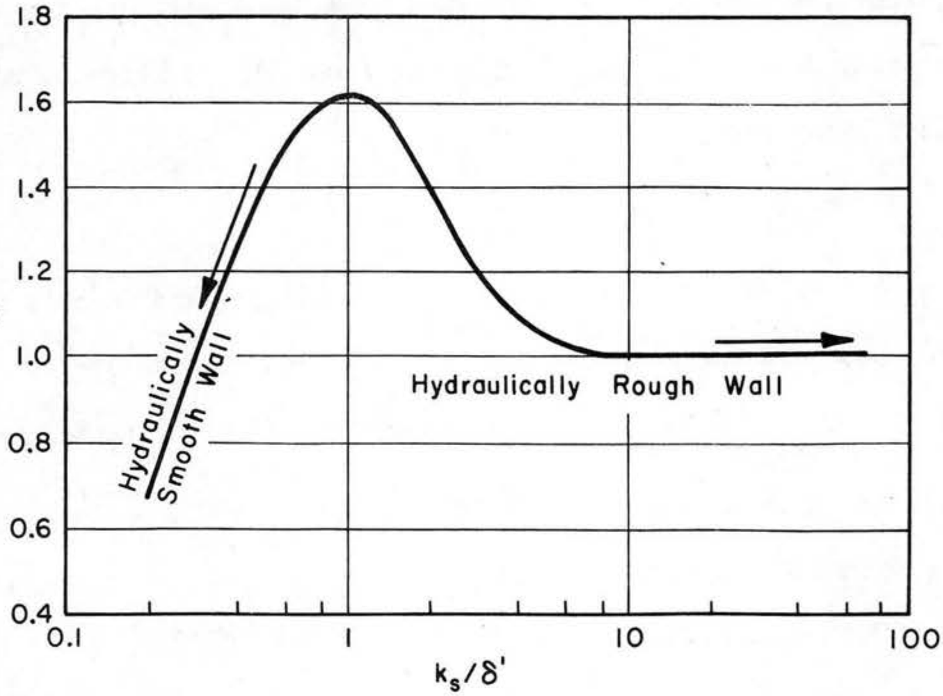


Figure 2-8 Einstein's Multiplication Factor x in the Logarithmic Velocity Equations (after Einstein, 1950).

in rivers has generally been relied on. Two such empirical equations are in common use. They are the Manning equation

$$V = \frac{1.486}{n} R^{2/3} S_f^{1/2} \tag{2.31}$$

and the Chezy equation

$$V = CR^{1/2} S_f^{1/2} \tag{2.32}$$

where V = the average velocity in the cross section

n = Manning's roughness coefficient

R = the hydraulic radius equal to the cross-sectional area A divided by the wetted perimeter P

S_f = the energy slope of the channel

C = Chezy's discharge coefficient

In these equations, the boundary shear stress is expressed implicitly in the roughness coefficient n or in the discharge coefficient C.

By equating the velocity determined from Manning's equation with the velocity determined from Chezy's equation, the relation between the coefficients is

$$C = \frac{1.486}{n} R^{1/6} \tag{2.33}$$

If the flow is gradually varied, Manning's and Chezy's equations are used with the energy slope S_f replaced with an average friction slope $S_{f_{ave}}$. The term $S_{f_{ave}}$ is determined by averaging over a short time increment at a station or over a short length increment at an instant of time, or both.

Over many decades a catalog of values of Manning's n and Chezy's C has been assembled so that an engineer can estimate the appropriate value by knowing the general nature of the channel boundaries. An abbreviated list of Manning's roughness coefficients is given in Table 2-1. Additional values are given by Barnes (1967) and Chow (1959).

2.1.4.4 Average Boundary Shear Stress. The shear stress at the boundary τ_o for steady uniform flow is determined by applying the conservation of mass and momentum principles to the control volume shown in Figure 2-9. From the continuity equation (2.6)

$$V_1 A_1 = V_2 A_2 = Q$$

or in this case

$$V_1 = V_2$$

Applying the momentum equation (2.12) with the momentum coefficient assumed equal to unity

Table 2-1 Manning's Roughness Coefficients for Various Boundaries

Rigid Boundary Channels	Manning's n
Very smooth concrete and planed timber	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Wood	0.014
Vitrified clay	0.015
Shot concrete, untrowelled, and earth channels in best condition	0.017
Straight unlined earth canals in good condition	0.020
Rivers and earth canals in fair condition-some growth	0.025
Winding natural streams and canals in poor condition-considerable moss growth	0.035
Mountain streams with rocky beds	0.040-0.050
Alluvial Sandbed Channels (no vegetation)[1/]	
Tranquil flow, Fr < 1	
plane bed	0.014-0.020
ripples	0.018-0.030
dunes	0.020-0.040
washed out dunes or transition	0.014-0.025
plane bed	0.010-0.013
Rapid flow, Fr ≃ 1	
standing waves	0.010-0.015
antidunes	0.012-0.020

[1/] Data is limited to sand channels with $D_{50} < 1.0$ mm.

$$\rho V_2 y_o W V_2 - \rho V_1 y_o W V_1 = \sum F_x$$

The pressure force acting on the control boundary at the upstream section is

$$F_1 = \int_o^{y_o} p_1 W \, dy$$

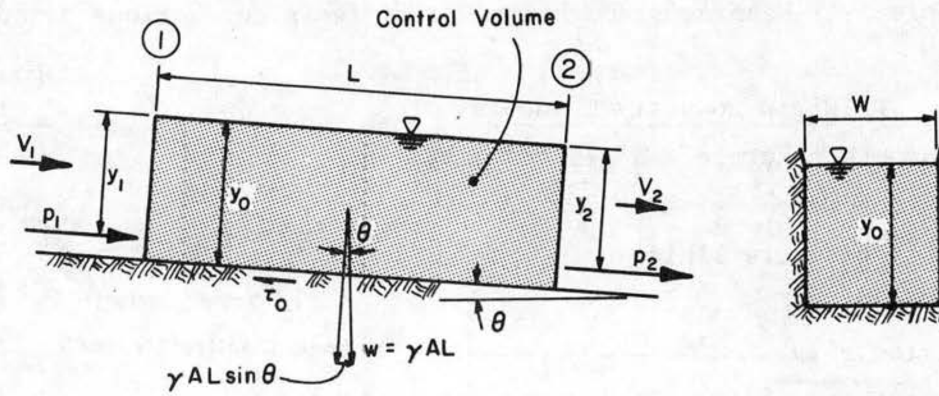


Figure 2-9 Control Volume for Steady Uniform Flow.

As the flowlines are parallel

$$p_1 = \gamma y_1$$

so

$$F_1 = \int_0^{y_0} \gamma W y \, dy = \frac{\gamma y_0^2 W}{2}$$

Similarly at the downstream section the force acting on the boundary is

$$F_2 = \frac{-\gamma y_0^2 W}{2}$$

The body force is the weight of the fluid in the control volume

$$\gamma AL$$

and the downstream component of this body force is

$$\gamma AL \sin\theta$$

where θ is the slope angle of the channel bed. The average boundary shear stress is τ_o acting on the wetted perimeter P. The shear force F_s in the x-direction is

$$F_s = \tau_o PL$$

With the above expressions for the components, the statement of conservation of linear momentum becomes

$$\rho V_2 y_o W\, V_2 - \rho V_1 y_o W\, V_1 = \gamma AL \sin\theta + \frac{\gamma y_o^2\, W}{2} - \frac{\gamma y_o^2\, W}{2} - \tau_o\, PL$$

which reduces to

$$\tau_o = \gamma \frac{A}{P} \sin\theta \qquad (2.34)$$

The term A/P is called the hydraulic radius R. If the channel slope angle is small

$$\sin\theta \approx S_o$$

and the average shear stress on the boundary is

$$\tau_o = \gamma R S_o \qquad (2.35)$$

If the flow is gradually varied flow, the average boundary shear stress is

$$\tau_o = \gamma R S_f \qquad (2.36)$$

where S_f is the slope of the energy grade line.

2.1.5 Hydraulics of Steady 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 2-10. The differential

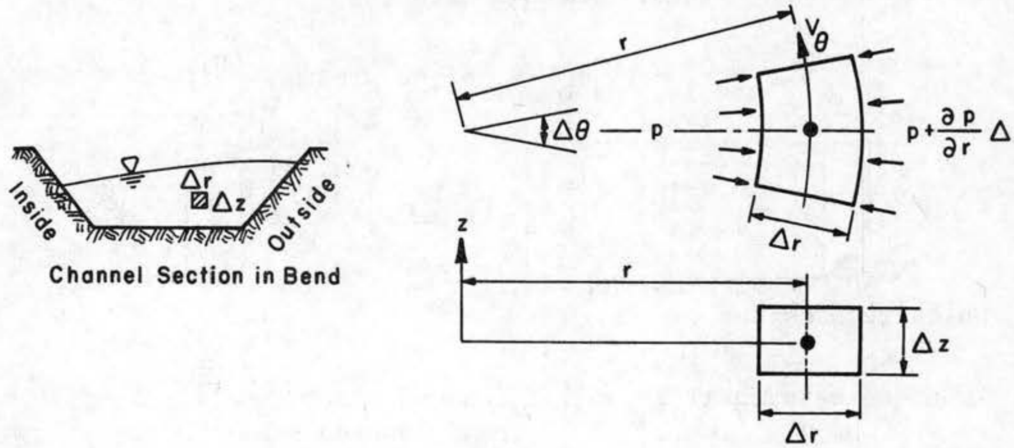


Figure 2-10 Definition Sketch of Dynamics of Flow Around a Bend.

pressure in the radial direction can be obtained by equating radial forces to the product of mass and radial acceleration:

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

$$\frac{1}{\rho}\,\frac{\partial p}{\partial r} = \frac{V_\theta^2}{r} \qquad (2.37)$$

The total superelevation between outer and inner bank is

$$\Delta z = \frac{1}{\rho g}\int_{p_i}^{p_o} dp = \frac{1}{g}\int_{r_i}^{r_o} \frac{V_\theta^2}{r}\,dr \qquad (2.38)$$

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

$$\Delta z = \frac{1}{g}\int_{r_i}^{r_o} \frac{V^2}{r}\,dr \qquad (2.39)$$

Equation (2.39) 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

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

$$\Delta z = z_o - z_i = \frac{V^2}{g r_c} (r_o - r_i) \tag{2.40}$$

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 (2.40) 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 2-11a, 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 2-11b, 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

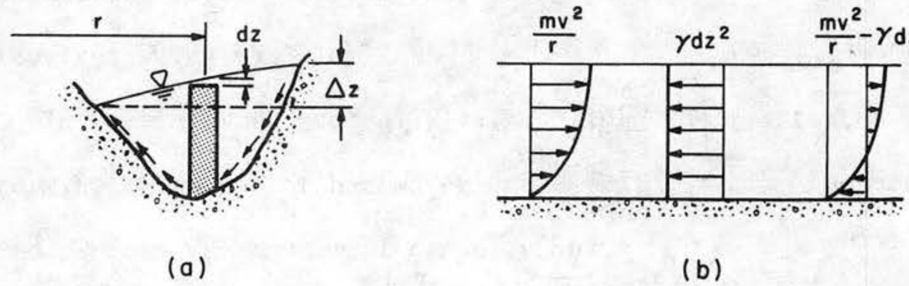


Figure 2-11 Schematic Representation of Transverse Currents in a Channel Bend.

these two curves (Figure 2-11b) 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.

2.1.6 Steady Gradually Varied Flow

2.1.6.1 Introduction. In uniform flow, acceleration forces are zero and energy is converted to heat as a result of viscous forces within the flow; there are no changes in cross section or flow direction and the depth (called normal depth) is constant. In steady uniform flow, the slope of the bed, the slope of the water surface and the slope of the energy gradeline are all parallel and are equal to the head loss divided by the length of channel in which the loss occurred. Steady gradually varied flow is nonuniform in that changes in depth and velocity take place slowly over large distances. Resistance to flow dominates and acceleration forces are neglected. The study of this type of flow involves: 1) the determination of the general characteristics of the water surface and 2) the elevation of the water surface or depth of flow.

In gradually varied flow, the actual flow depth y is either larger than or smaller than the normal depth y_o and either larger than or smaller than the critical depth y_c. The water surface profiles which are often called backwater curves, depend on the magnitude of the actual depth of flow y in relation to the normal depth y_o and the critical depth y_c. Normal depth y_o is the depth of flow that would exist for uniform flow as determined from the Manning or Chezy equation and the critical depth is the depth of flow when the Froude number equals 1.0.

The Froude number has been defined (Section 2.1.2) as the ratio of inertia forces in the system to gravitational forces, or

$$Fr = \frac{V}{\sqrt{gy}}$$

Flow at the critical depth, y_c, implies that the local flow velocity, V, is equal to the velocity of propagation of a small gravity wave, $\sqrt{gy}$, on the free surface.

Flow depth can be different from the normal depth because of chang in slope of the bed, changes in cross section, obstruction to flow, and imbalances between gravitational forces accelerating the flow and shear forces retarding the flow.

In working with gradually varied flow, the first step is to determine what type of backwater curve would exist. The second step is to perform the numerical computations.

2.1.6.2 Classification of Flow Profiles. The classification of flow profiles is obtained by analyzing the change of the various terms in the total head equation in the x-direction. The total head is

$$H_T = \frac{V^2}{2g} + y + z \tag{2.41}$$

or

$$H_T = \frac{Q^2}{2gA^2} + y + z \tag{2.42}$$

Then assuming a wide channel for simplicity

$$\frac{dH_T}{dx} = - \frac{q^2}{gy^3} \frac{dy}{dx} + \frac{dy}{dx} + \frac{dz}{dx} \tag{2.43}$$

where the flow per unit width is

$$q = Vy \tag{2.44}$$

The term $-dH_T/dx$ is the slope of the energy gradeline, S_f. For short distances and small changes in y the energy gradient can be evaluated using Manning's or Chezy's equation.

When Chezy's equation (2.32) is used, the expression for dH_T/dx is

$$- \frac{dH_T}{dx} = S_f = \frac{q^2}{C^2 y^3} \tag{2.45}$$

The term $-dy/dx$ is the slope of the water surface S_w and $-dz/dx$ is the bed slope S_o. For steady uniform flow, the bed slope is (from Equation (2.32))

$$S_o = \frac{q^2}{C_o^2 y_o^3} \tag{2.46}$$

where the subscript "o" indicates the steady uniform flow values.

When Equations (2.45 and 2.46) are substituted into Equation (2.43), the familiar form of the gradually varied flow equation results

$$\frac{dy}{dx} = S_o \left\{ \frac{1 - (\frac{C_o}{C})^2 \ (\frac{y_o}{y})^3}{1 - (\frac{y_c}{y})^3} \right\} \tag{2.47}$$

If Manning's equation is used to evaluate S_f and S_o, Equation (2.47) becomes

$$\frac{dy}{dx} = S_o \left\{ \frac{1 - (\frac{n}{n_o})^2 \ (\frac{y_o}{y})^{10/3}}{1 - (\frac{y_c}{y})^3} \right\} \tag{2.48}$$

The slope of the water surface $\frac{dy}{dx}$ depends on the slope of the bed S_o, the ratio of the normal depth y_o to the actual depth y and the ratio of the critical depth y_c to the actual depth y. The difference between flow resistance for steady uniform flow n_o and flow resistance for steady nonuniform flow n is small and the ratio is taken as 1.0. With $n = n_o$, there are twelve types of water surface profiles. These are summarized in Table 2-2 and illustrated in Figure 2-12. When $y \to y_c$, the assumption that acceleration forces can

Table 2-2 Characteristics of Water Surface Profiles

Class	Bed Slope	Depth	Type	Classification
Mild	$S_o>0$	$y>y_o>y_c$	1	M_1
Mild	$S_o>0$	$y_o>y>y_c$	2	M_2
Mild	$S_o>0$	$y_o>y_c>y$	3	M_3
Critical	$S_o>0$	$y>y_o=y_c$	1	C_1
Critical	$S_o>0$	$y<y_o=y_c$	3	C_3
Steep	$S_o>0$	$y>y_c>y_o$	1	S_1
Steep	$S_o>0$	$y_c>y>y_o$	2	S_2
Steep	$S_o>0$	$y_c>y_o>y$	3	S_3
Horizontal	$S_o=0$	$y>y_c$	2	H_2
Horizontal	$S_o=0$	$y_c>y$	3	H_3
Adverse	$S_o<0$	$y>y_c$	2	A_2
Adverse	$S_o<0$	$y_c>y$	3	A_3

be neglected no longer holds. Equations (2.47 and 2.48) indicate that $\frac{dy}{dx}$ is perpendicular when $y \to y_c$. For sections close to the cross section where the flow is critical (a distance from 10 to 50 feet) curvilinear flow analysis and experimentation must be used to determine the actual values of y. When analyzing long distances (100 to 1000 feet or longer) one can assume qualitatively that y reaches y_c. In general when the flow is rapid ($Fr \geq 1$), the flow cannot become tranquil without

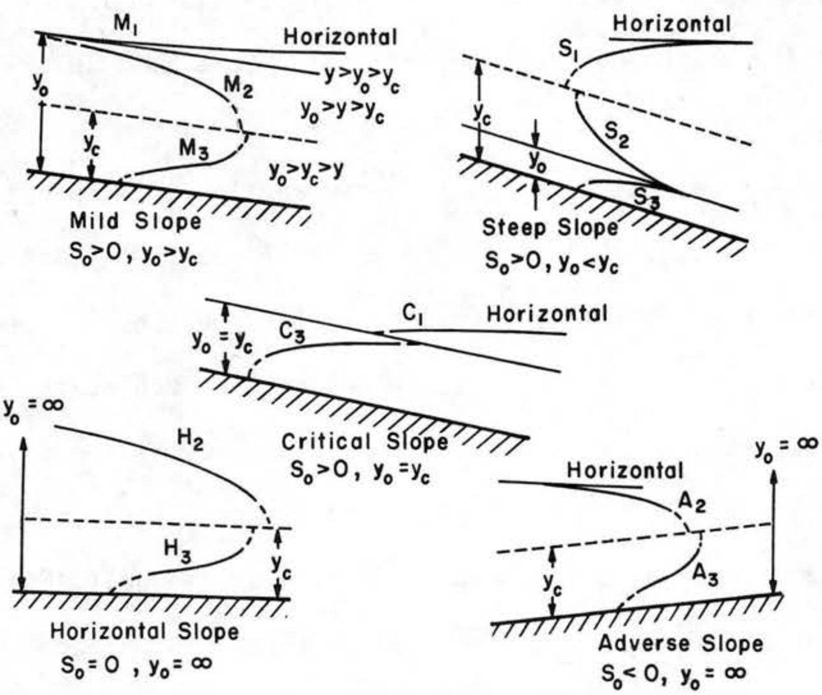


Figure 2-12 Classification of Water Surface Profiles.

Note:

(1) With a type 1 curve (M_1, S_1, C_1), the actual depth of flow y is greater than both the normal depth y_o and the critical depth y_c. Because flow is tranquil, control of the flow is downstream.

(2) With a type 2 curve (M_2, S_2, A_2, H_2), the actual depth y is between the normal depth y_o and the critical depth y_c. The flow is tranquil for M_2, A_2 and H_2 and thus the control is downstream. Flow is rapid for S_2 and the control is upstream.

(3) With a type 3 curve (M_3, S_3, C_3, A_3, H_3), the actual depth y is smaller than both the normal depth y_o and the critical depth y_c. Because the flow is rapid control is upstream.

(4) For a mild slope S_o is smaller than S_c and $y_o > y_c$.

(5) For a steep slope, S_o is larger than S_c and $y_o < y_c$.

(6) For a critical slope, S_o equals S_c and $y_o = y_c$.

(7) For an adverse slope, S_o is negative.

(8) For a horizontal slope, S_o equals zero.

(9) The case where $y \to y_c$ is of special interest because the denominator in Equations (2.47 and 2.48) approaches zero.

a hydraulic jump occurring. In contrast, tranquil flow can become rapid (cross the critical depth line). This is illustrated in Figure 2-13.

When there is a change in cross section or slope at an obstruction to the flow, the qualitative analysis of the flow profile depends on locating the control points, determining the type of curve upstream and downstream of the control points and then sketching the backwater curves. It must be remembered that when flow is rapid ($Fr \geq 1$) the control of the depth is upstream and the backwater calculations proceed in the downstream direction. When flow is tranquil ($Fr < 1$) the depth control is downstream and the computations must proceed upstream. The backwater curves that result from a change in slope of the bed are illustrated in Figure 2-13.

2.1.6.3 Computation of Water Surface Profiles. There are many computer programs available for the computation of the elevation or depth of flow for water surface profiles. Herein, the standard step method is described. However, as with most computer programs, a qualitative analysis of the general characteristics of the backwater curves as described in the preceding section must be made. This is necessary in order to know whether the analysis proceeds upstream or downstream. Most available computer programs cannot solve the water surface profile equations when the flow changes from rapid to tranquil or vice versa.

The standard step method is derived from the energy equation (2.18

$$\frac{V_1^2}{2g} + y_1 + \Delta z = \frac{V_2^2}{2g} + y_2 + H_L$$

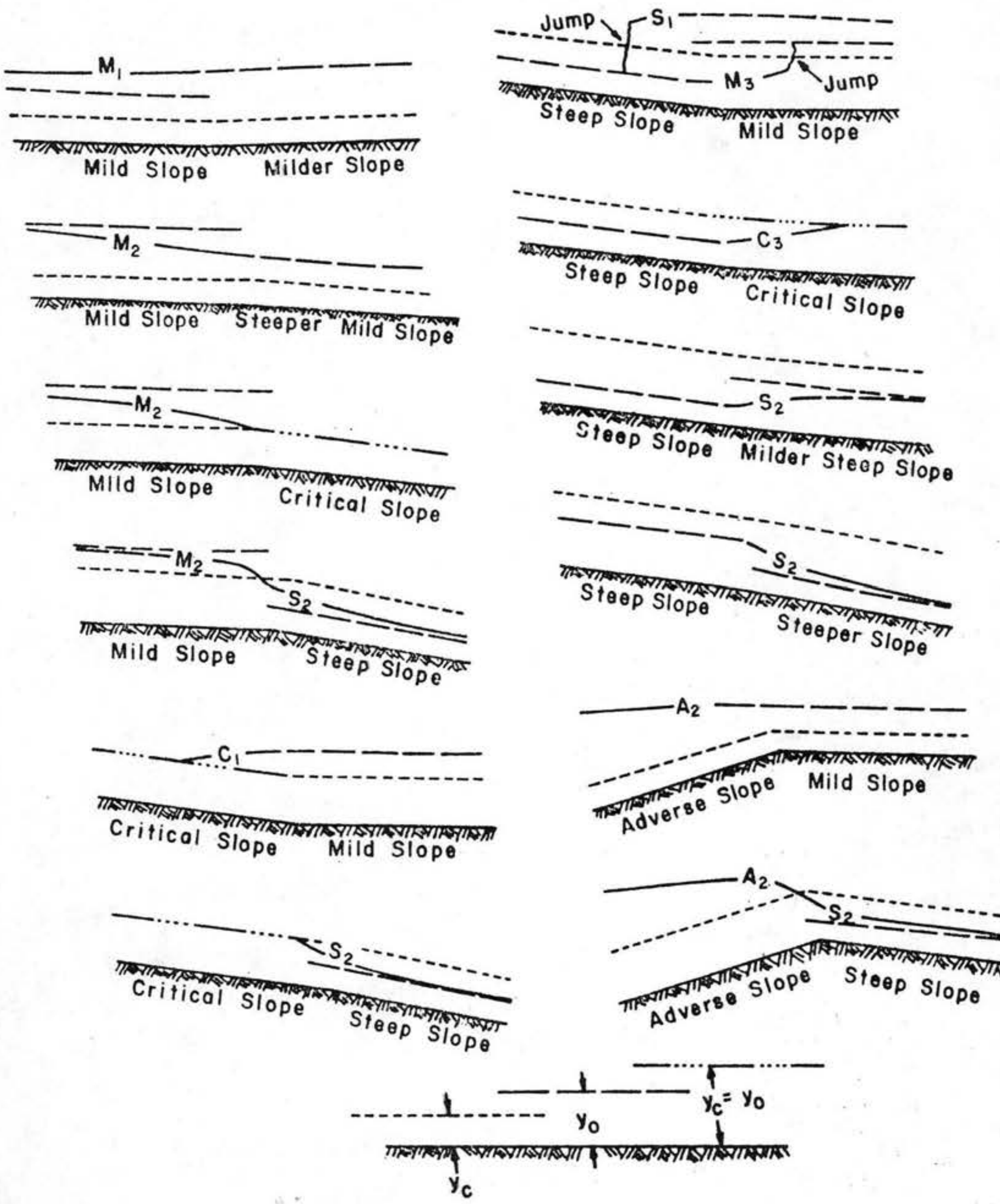


Figure 2-13 Examples of Water Surface Profiles.

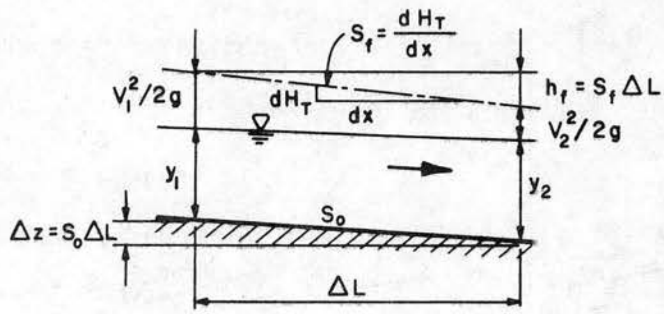


Figure 2-14 Definition Sketch for Step Method Computation of Backwater Curves.

From Figure 2-14

$$\frac{V_1^2}{2g} + y_1 + S_o \Delta L = \frac{V_2^2}{2g} + y_2 + S_f \Delta L \tag{2.49}$$

$$H_1 + S_o \Delta L = H_2 + S_f \Delta L \tag{2.50}$$

and

$$\Delta L = \frac{H_2 - H_1}{S_o - S_f} \tag{2.51}$$

The procedure for computing water surface profiles is to start from some known y, assume another y either upstream or downstream depending on whether the flow is tranquil or rapid, and compute the distance ΔL to the assumed depth using Equation (2.51).

2.1.7 Steady Rapidly Varied Flow

2.1.7.1 Introduction. Steady flow through relatively short transitions where the flow is uniform before and after the transition can be analyzed using the energy equation (2.18). Energy loss due to friction may be neglected; at least as a first approximation. Refinement of the analysis can be made as a second step by including friction loss. For example, the water surface elevation through a transition is determined using the Bernoulli equation and then modified by determining the friction loss effects on velocity and depth in short reaches through the transition. Energy losses resulting from separation cannot be neglected and transitions where separation may occur need special treatment which may include model studies. Contracting flows (converging streamlines) are less susceptible to separation than expanding flows. Also, any time a transition changes velocity and depth such that the Froude number approaches unity, problems such as waves, and blockage or choking of the flow may occur. If the approaching flow is rapid (supercritical), a hydraulic jump may result.

Transitions are used to contract or expand channel width, (Figure 2-15a); to increase or decrease bottom elevation (Figure 2-15b);

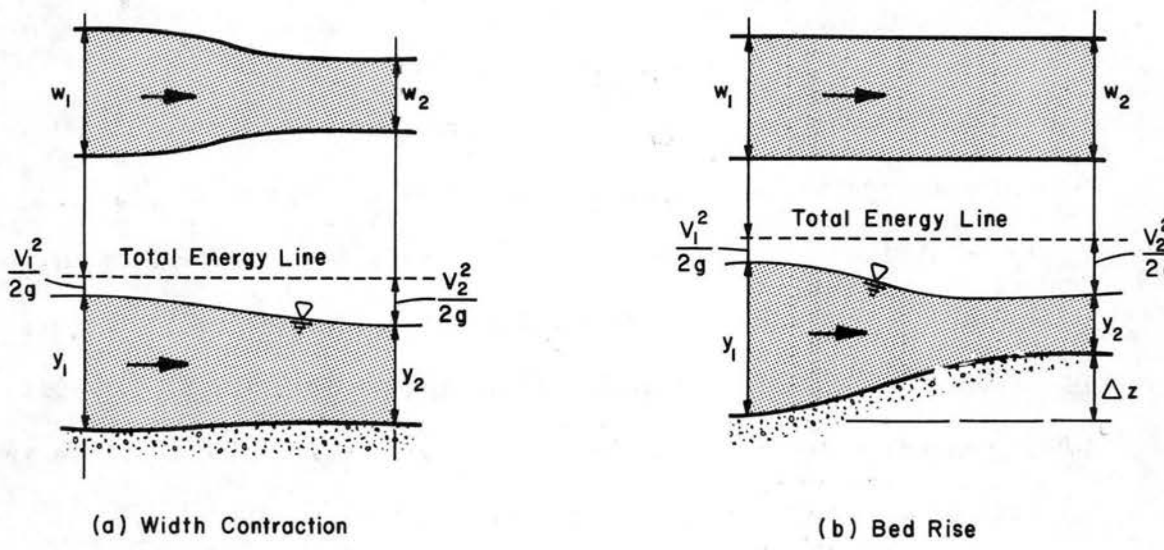


Figure 2-15 Transitions in Open Channel Flow.

or to change both the width and bottom elevation. The first step in the analysis is to use the Bernoulli equation (neglecting any head loss resulting from friction or separation) to determine the depth and velocity changes of the flow through the transition. Further refinement depends on importance of freeboard, whether flow is rapid, and whether flow approaches critical.

The Bernoulli equation for flow in Figure 2-15b is

$$\frac{V_1^2}{2g} + y_1 = \frac{V_2^2}{2g} + y_2 + \Delta z \tag{2.52}$$

or

$$H_1 = H_2 + \Delta z \tag{2.53}$$

where

$$H = \frac{V^2}{2g} + y \tag{2.54}$$

The term H is called the specific head and is the height of the total head above the channel bed.

In the analysis of flow through transitions, the Bernoulli equation gives a numerical solution to the problem but very little descriptive information of the depth variation. Only after the analysis is completed will it be known if the depth will increase or decrease as the fluid passes through the transition or even if the flow is physically possible. On the other hand by investigating the various interrelations between the variables (H, V and y) in the specific head equation the variation of depth through a transition can be predicted.

There are two conditions for analyzing the flow through transitions. In the first condition, the width is constant and the elevation of the stream bed changes; that is, $q = Q/W$ is constant and H and y vary (Figure 2-15b). In the second, the width changes and the elevation of the stream bed (neglecting the slope) is constant; that is, H is constant (Figure 2-15a). These conditions can be qualitatively analyzed using a plot of depth versus specific head, called a specific head diagram.

2.1.7.2 Specific Head Diagram. For simplicity, the following specific head analysis is done on a unit width of channel so that Equation (2.54) becomes

$$H = \frac{q^2}{2gy^2} + y \tag{2.55}$$

For a given q, Equation (2.55) can be solved for various values of H and y. When y is plotted as a function of H, Figure 2-16 is

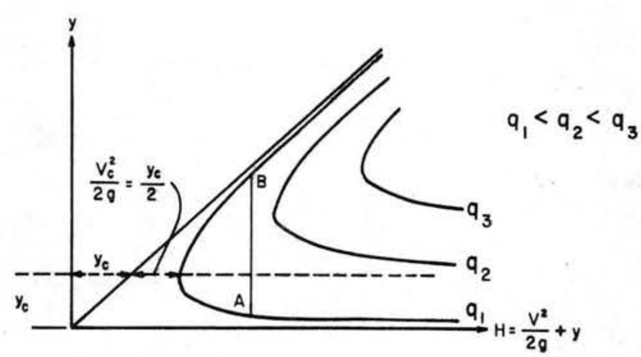


Figure 2-16 Specific Head Diagram.

obtained. There are two possible depths called alternate depths for any H larger than a specific minimum. Thus, for specific head larger than the minimum, the given flow may have a large depth and small velocity or small depth and large velocity. Flow cannot occur with specific energy less than the minimum. The single depth of flow at the minimum specific head is called the critical depth y_c and the corresponding velocity, the critical velocity $V_c = q/y_c$. To determine y_c the derivative of H with respect to y is set equal to 0.

$$\frac{dH}{dy} = -\frac{q^2}{gy^3} + 1 = 0 \tag{2.56}$$

and

$$q = (gy_c^3)^{1/2} \tag{2.57}$$

or

$$y_c = (\frac{q^2}{g})^{1/3} = 2\frac{V_c^2}{2g} \tag{2.58}$$

Note that

$$V_c^2 = y_c g \tag{2.59}$$

or

$$\frac{V_c}{\sqrt{gy_c}} = 1 \tag{2.60}$$

But

$$\frac{V}{\sqrt{gy}} = Fr \tag{2.61}$$

Also

$$H_{min} = \frac{V_c^2}{2g} + y_c = \frac{3}{2}y_c \tag{2.62}$$

Thus flow at minimum specific energy has a Froude number equal to one. Flows with velocities larger than critical ($Fr > 1$) are called rapid or supercritical and flows with velocities smaller than critical ($Fr < 1$) are called tranquil or subcritical. These flow conditions are illustrated in Figure 2-17 where a rise in the bed causes a decrease in dept when the flow is tranquil and an increase in depth when the flow is rapid. Furthermore there is a maximum rise in the bed for a given H_1 where the given rate of flow is physically possible. If the rise in th

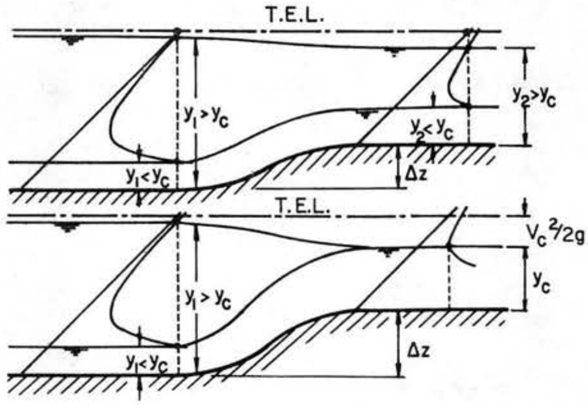


Figure 2-17 Changes in Water Surface Resulting from an Increase in Bed Elevation.

bed is increased beyond Δz_{max} for H_{min} then the approaching flow depth y_1 would have to increase (increasing H) or the flow would have to be decreased. Thus, for a given flow in a channel, a rise in the bed level can occur up to a Δz_{max} without causing backwater.

2.1.7.3 Specific Discharge Diagram. For a constant H, Equation (2.55) can be solved for y as a function of q. By plotting y as a function of q, Figure 2-18 is obtained. For any discharge

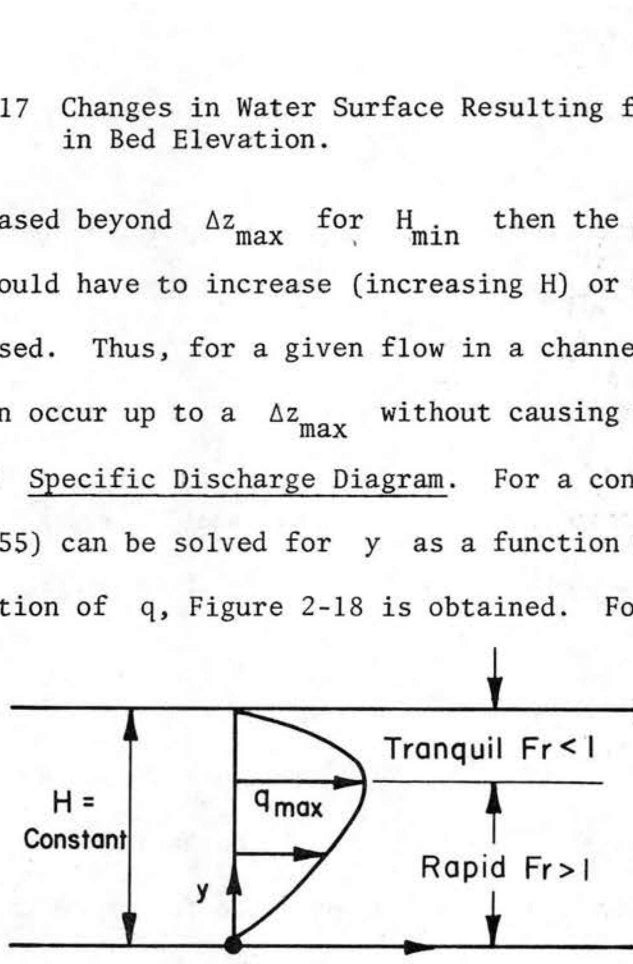


Figure 2-18 Specific Discharge Diagram.

smaller than a specific maximum, two depths of flow are possible. To determine the value of y for q_{max} Equation (2.55) is rearranged to obtain

$$q = y\sqrt{2g(H-y)} \tag{2.63}$$

The differential with respect to y is set equal to zero.

$$\frac{dq}{dy} = 0 = \frac{\sqrt{g}}{2} \frac{(2H-3y)}{(H-y)^{1/2}} \tag{2.64}$$

from which

$$y_c = \frac{2}{3} H = 2\frac{V_c^2}{2g} \tag{2.65}$$

or

$$V_c = \sqrt{gy_c} \tag{2.66}$$

Thus for maximum discharge at constant H, the Froude number is 1.0 and the flow is critical. From this

$$y_c = \frac{2}{3} H = 2 \frac{V_c^2}{2g} = (\frac{q_{max}^2}{g})^{1/3} \tag{2.67}$$

For critical conditions, the Froude number is 1.0, the discharge is a maximum for a given specific head, and the specific head is a minimum for a given discharge.

Flow conditions at constant specific head for a width contraction are illustrated in Figure 2-19. The contraction causes a decrease in flow depth when the flow is tranquil and an increase when the flow is rapid. The maximum contraction possible for these flow conditions is to the critical depth. Then the Froude number is one, the discharge per unit width q is a maximum, and y_c is $\frac{2}{3}$ H. A further decrease in width causes backwater, that is, an increase in y_1 upstream to get a larger specific energy and increase y_c in order to get the flow through the decreased width.

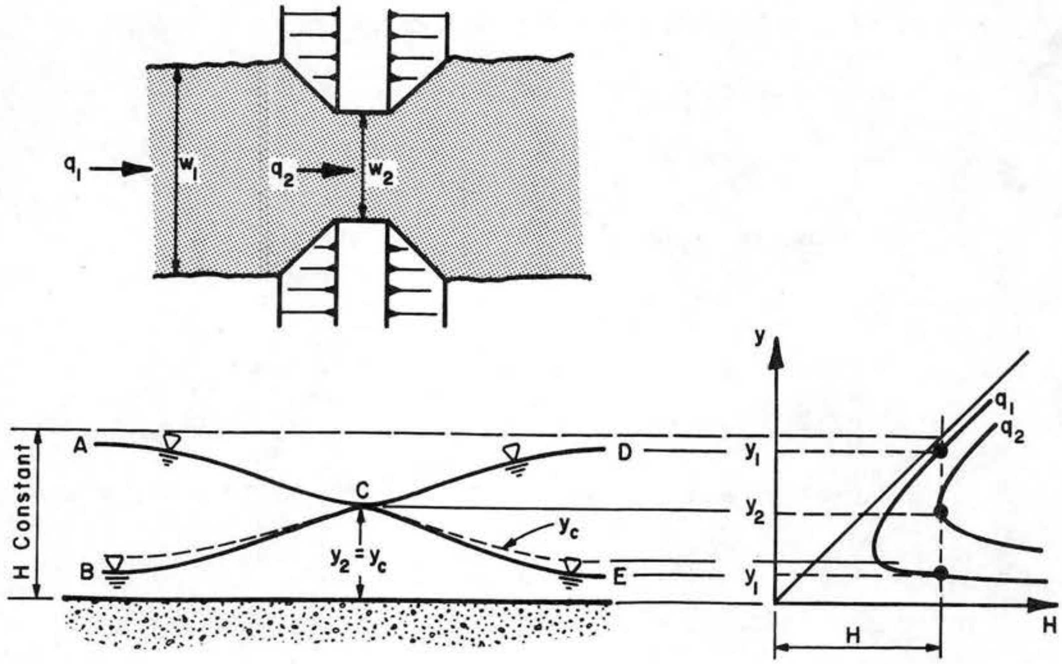


Figure 2-19 Change in Water Surface Elevation Resulting from a Change in Width.

The flow in Figure 2-19 can go from point A to C and then back to D or down to E depending on the boundary conditions. An increase in slope of the bed downstream from C and no separation would allow the flow to follow the line A to C to E. Similarly the flow can go from B to C and back to E or up to D depending on boundary conditions.

Hydraulic concepts have been presented in preceeding sections with the purpose of providing exposure to the basic definitions and principles of open channel flow. The discussion has included those aspects of steady uniform flow, steady gradually varied flow, and steady rapidly varied flow which are considered essential to an understanding of any rigid boundary, open channel flow problem, and which provide the necessary background for an appreciation of the more complex problems of mobile boundary hydraulics. For a more detailed treatment of this subject as well as more advanced topics such as unsteady flow concepts reference to such standard works as Chow (1959) or Henderson (1966) is suggested.

2.2 Flow in Alluvial Channels

2.2.1 Introduction

Most streams flow on sandbeds for the greater part of their length and nearly all large rivers have sandbeds. In sandbed rivers, the bed material is easily eroded and is continually being moved and shaped by the flow. The interaction between the flow of the water-sediment mixture and the sandbed creates different bed configurations which change the resistance to flow and rate of sediment transport. The gross measures of channel flow such as the flow depth, river stage, bed elevation and flow velocity change with different bed configurations. In the extreme case, the change in bed configuration can cause a three-fold change in resistance to flow and a 10-to-15 fold change in concentration of bed-material transport. For a given discharge and channel width, a three-fold increase in Manning's n results in a doubling of the flow depth.

The interaction between the flow and bed material and the interdependency among the variables makes the analysis of flow in alluvial sandbed streams extremely complex. However, with an understanding of the different types of bed forms that may occur and a knowledge of the resistance to flow and sediment transport associated with each bed form, alluvial channel flow can be analyzed.

2.2.2 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, 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:

$$\phi[V, y, S, \rho, \mu, g, D, \sigma, \rho_s, S_p, S_R, S_C, f_s, C_T, C_F, \omega] = 0 \qquad (2.68)$$

in which

V = velocity

y = 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, y, and ρ selected as repeating variables, yields:

$$\phi_1\left[S, \frac{Vy\rho}{\mu}, \frac{V}{\sqrt{gy}}, \frac{D}{y}, \sigma, \frac{\rho_s}{\rho}, S_p, S_R, S_C, \frac{f_s}{\rho V^2}, C_T, C_F, \frac{\omega}{V}\right] = 0 \qquad (2.69)$$

Equation (2.69) provides a list of nondimensional parameters which are important in a study of alluvial channel characteristics. These include the Froude number $(V/\sqrt{gy})$, the Reynolds number $(Vy\rho/\mu)$ and a relative roughness parameter (D/y).

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.

2.2.3 Bed Forms and Resistance

2.2.3.1 Introduction. 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 .

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 $\gamma y_o SV$) 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.

The different forms of bed-roughness are not mutually exclusive in time and space in a stream. Bed-roughness elements may form side-by-side in a cross section or reach of a natural stream giving a multiple roughness; or they may form in time sequence producing variable roughness.

Multiple roughness is related to variations in shear stress ($\gamma y_o S$) and stream power ($V\gamma y_o S$) in a channel cross section. The greater the width-depth ratio of a stream, the greater is the probability of a spatial variation in shear stress, stream power or bed material. Thus, the occurrence of multiple roughness is closely related to the width-depth ratio of the stream.

Variable roughness is related to changes in shear stress, stream power, or reaction of bed material to a given stream power over time. A commonly observed example of the effect of changing shear stress or stream power is the change in bed form that occurs with changes in depth during a runoff event. Another example is the change in bed form that occurs with change in the viscosity of the fluid as the temperature or concentration of fine sediment varies over time. It should be noted that a transition occurs between the dune bed and the plane bed; either bed configuration may occur for the same value of stream power.

2.2.3.2 Bed Configuration without Sediment Movement. If the bed material of a stream moves at one discharge but not at a smaller discharge, the bed configuration at the smaller discharge will be a remnant of the bed configuration formed when sediment was moving. Prior to the beginning of motion, the problem of resistance to flow is one of rigid-boundary hydraulics. After beginning of motion, the problem relates to defining bed configuration and resistance to flow.

Plane bed without movement has been studied to determine the flow conditions for the beginning of motion and the bed configuration that would form after beginning of motion. In general, Shields' relation, Figure 2-20, for the beginning of motion is adequate. After the

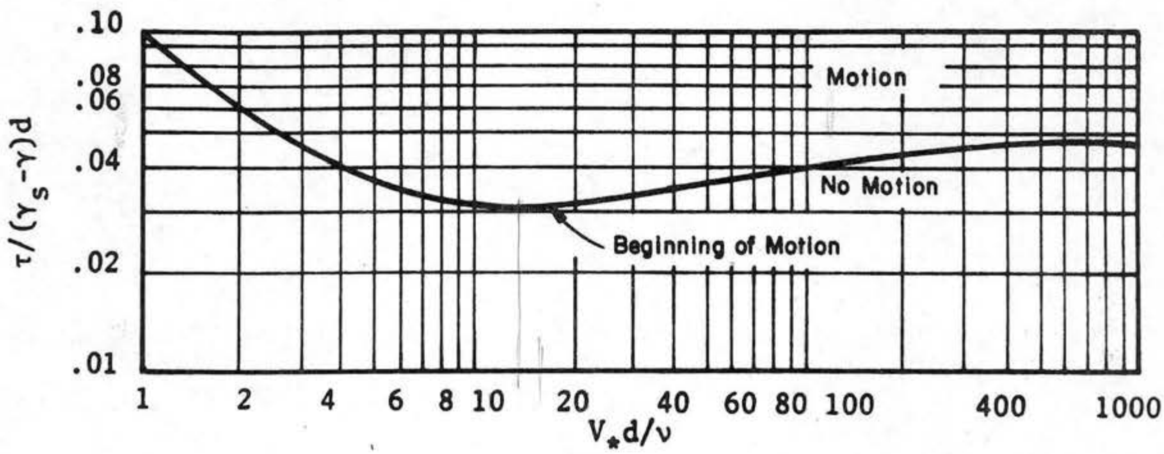


Figure 2-20 Shields Relation for Beginning of Motion (after Gessler, 1971).

beginning of motion, for flat slopes and low velocity, the plane bed will change to ripples for sand material smaller than 0.6 mm, and to dunes for coarser material. Resistance to flow is small for a plane bed without sediment movement and is due solely to the sand grain roughness. Values of Manning's n range from 0.012 to 0.014 depending on the size of the bed material.

2.2.3.3 Bed Configuration with Sediment Movement. Typical bed forms and their relation to the water surface (in phase or out of phase) are shown in Figures 2-21 and 2-22. Using these bed forms as a basic

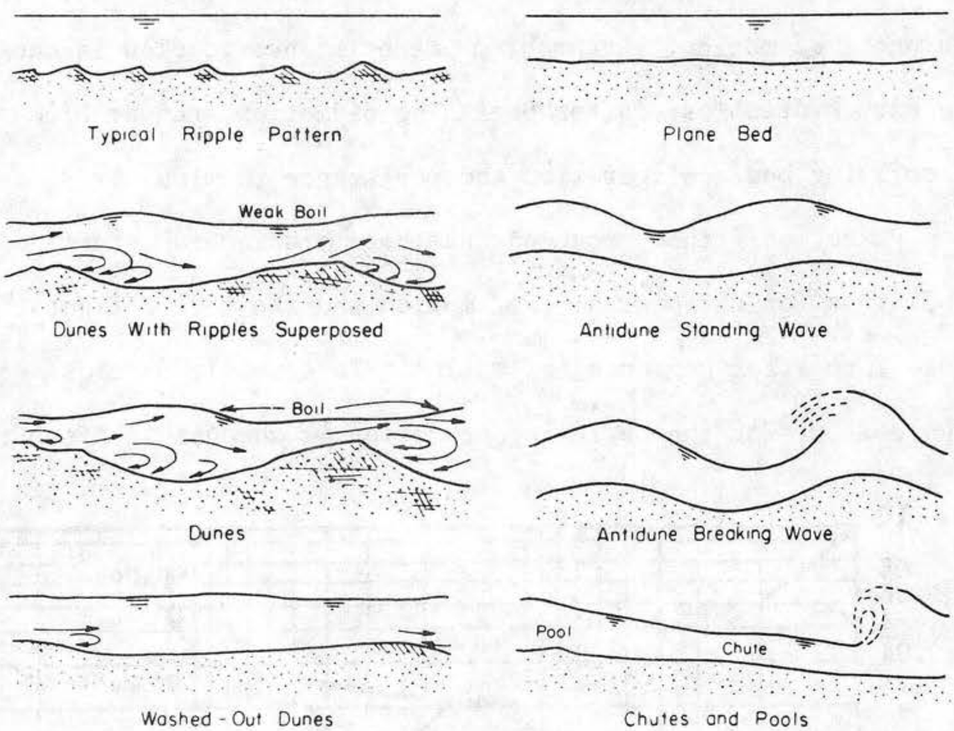


Figure 2-21 Forms of Bed Roughness in Sand Channels.

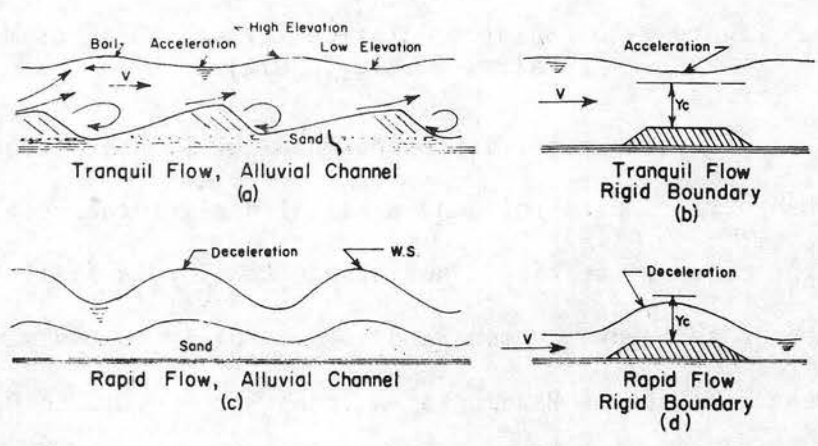


Figure 2-22 Relation between Water Surface and Bed Configuration.

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. 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 2-23) gives 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 2-24) 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

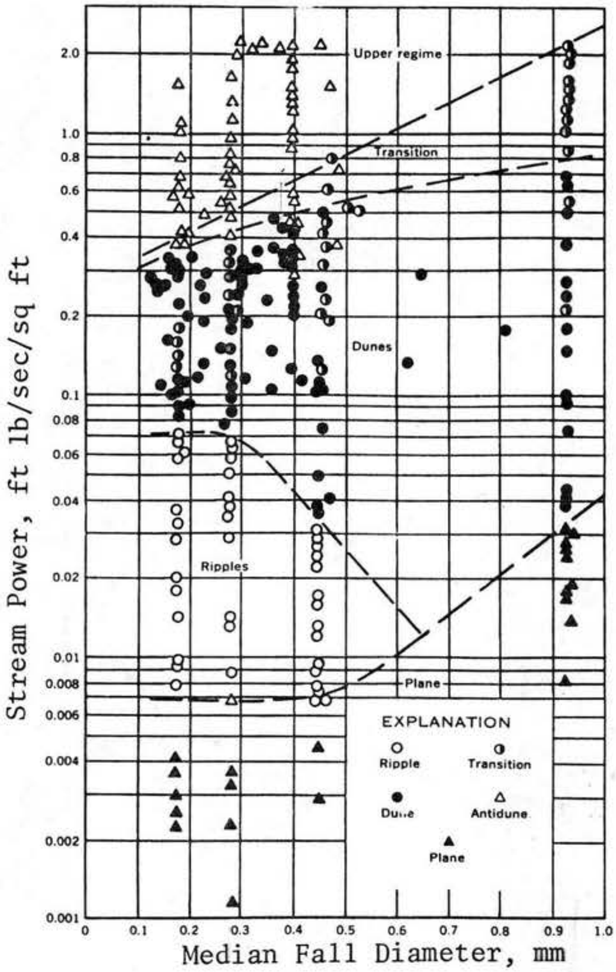


Figure 2-23 Relation of Stream Power and Median Fall Diameter to Form of Bed Roughness.

bed without sediment motion to values as high as 0.04 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 0.010 to 0.015. The consequent effect on flow velocity can be seen in Figure 2-25. This data pertains to a single sand size (D_{50} = 0.19 mm) and was determined in the 8-foot flume at Colorado State University.

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 2-26 shows a typical break in the depth-discharge relation resulting from this phenomena. Conversely, several investigators have shown that an increase

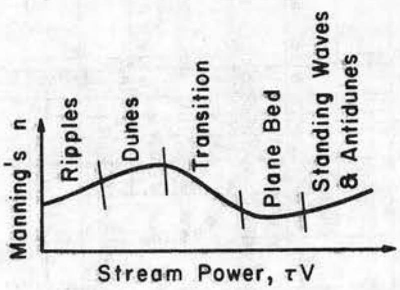


Figure 2-24 Resistance to Flow versus Bed Form.

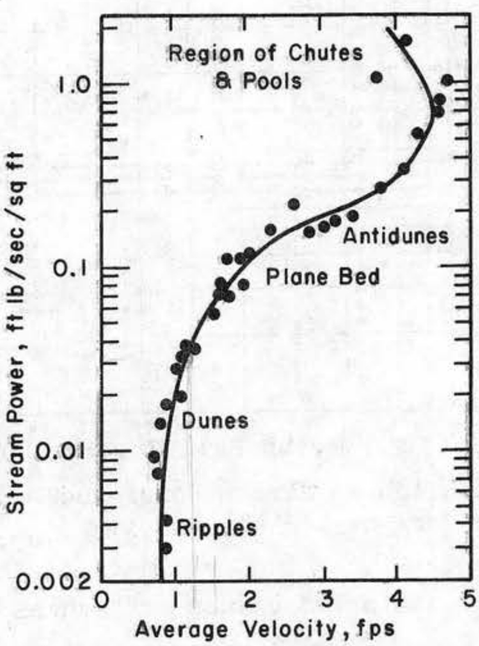


Figure 2-25 Relation of Bed Form to Stream Power and Velocity.

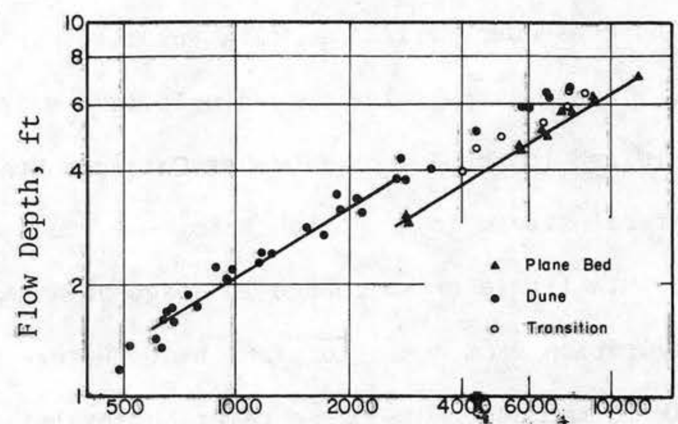


Figure 2-26 Relation of Depth to Discharge for Elkhorn River near Waterloo, Nebraska (after Simons and Richardson, 1971).

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.

2.2.3.4 Bars in Alluvial Channels. In natural or field-size channels, other bed configurations are also found. These bed configurations are generally called bars and are related to the plan form geometry and the width of the channel.

Bars are bed forms having lengths of the same order as the channel width or greater, and heights comparable to the mean depth of the generating flow. Several different types of bars occur. They are classified as:

(1) Point Bars which occur adjacent to the convex bank of channel bends. Point bar shape may vary with changing flow conditions, but point bars do not move relative to the bends.

(2) Alternate Bars which occur in straighter reaches of channels and tend to be distributed periodically along the reach, with consecutive bars on opposite sides of the channel. Their lateral extent is significantly less than the channel width. Alternate bars move slowly downstream.

(3) Transverse Bars (middle bars) which also occur in straight channels. They occupy nearly the full channel width. They occur both as isolated and as periodic forms along a channel, and move slowly downstream.

(4) Tributary Bars which occur immediately downstream from points of lateral inflow into a channel.

In longitudinal section, bars are approximately triangular, with very long gentle upstream slopes and short downstream slopes that are approximately the same as the angle of repose. Bars appear as small barren islands during low flows. Portions of the upstream slopes of bars are often covered with ripples or dunes.

2.2.3.5 Manning's "n" Values for Natural Sandbed Streams. Observations on natural sandbed streams with bed material having a median diameter ranging from 0.1 mm to 0.4 mm indicate that the bed planes out and resistance to flow decreases whenever high flow occurs. Manning's n changes from values as large as 0.040 at low flow to as small as 0.012 at high flow. An example is given in Figure 2-27. These

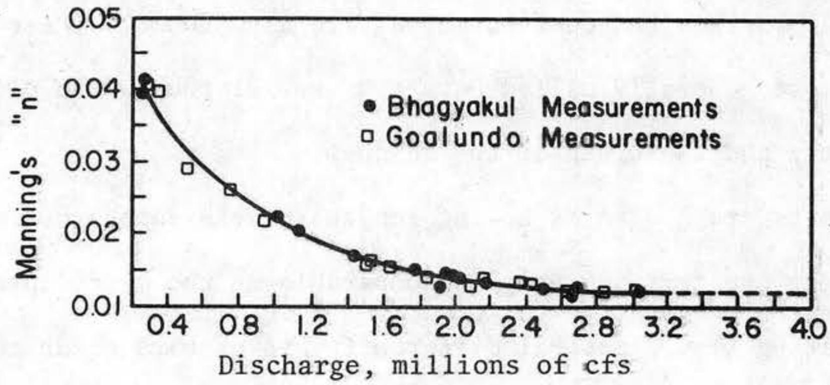


Figure 2-27 Change in Manning's n with Discharge for Padma River in Bangladesh.

observations are substantiated by Dawdy (1961), Colby (1960), Corps of Engineers (1968) and Beckman and Furness (1962).

The range in Manning's n for the various bed configurations is as follows:

	Lower flow regime		Upper flow regime
Ripples	$(0.018 \le n \le 0.028)$	Plane bed	$(0.010 \le n \le 0.013)$
Dunes	$(0.020 \le n \le 0.040)$	Antidunes	
		Standing waves	$(0.010 \le n \le 0.01$
		Breaking waves	$(0.012 \le n \le 0.0$
		Chute and pools	$(0.018 \le n \le 0.0$

2.3 Sediment Transport in Alluvial Channels

2.3.1 Introduction

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 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 river response to development.

2.3.2 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: 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 0.3 foot of the bed. 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:

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

Figure 2-28 shows the vertical distribution of suspended sediment for different particle size groups from one sediment sample.

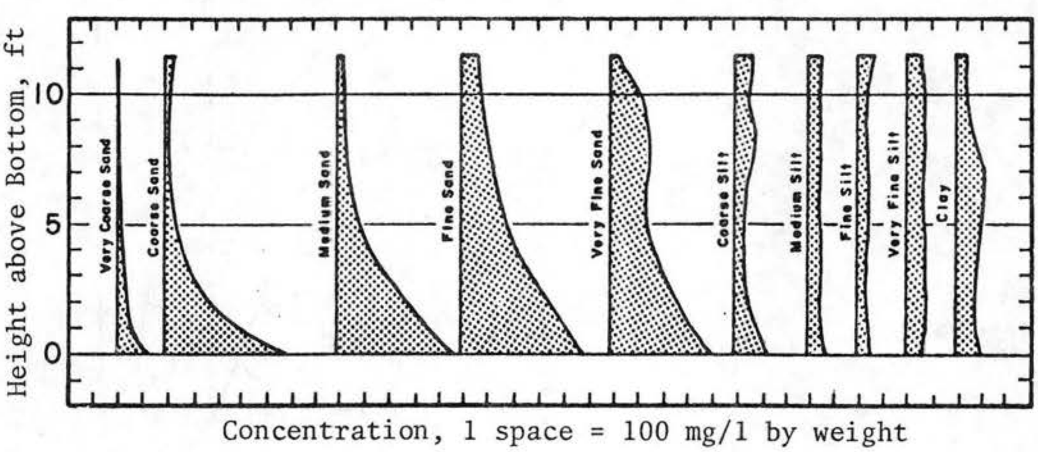


Figure 2-28 Discharge Weighted Concentration of Suspended Sediment for Different Particle Size Groups at a Sampling Vertical in the Missouri River (after Guy, 1970).

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

$$\frac{C}{C_a} = [\frac{y_o-y}{y} \frac{a}{y_o-a}]^z \qquad (2.70)$$

where

C = the concentration at a distance y from the bed

C_a = the concentration at a point a above the bed

y_o = the depth of flow

ω = particle fall velocity

β = a coefficient relating diffusion coefficients

κ = the von Karman velocity coefficient

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

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

Figure 2-29 shows a family of curves obtained by plotting Equation (2.70) for different values of the Rouse number z. It is apparent tha

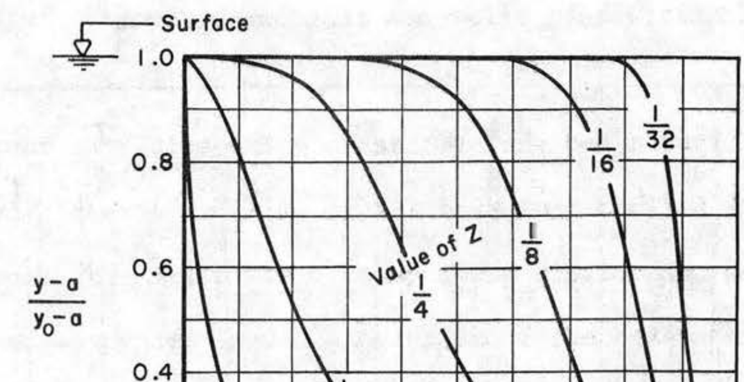


Figure 2-29 Graph of Suspended Sediment Distrubution.

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 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:

$$C = \left[\frac{\text{weight of sediment}}{\text{weight of water-sediment mixture}} \right] \qquad (2.71)$$

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

$$C \text{ in ppm} = (C \text{ by weight})(1{,}000{,}000) \qquad (2.72)$$

The particle fall velocity, ω, used in Equation (2.70) 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.

2.3.3 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 0.062 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 in suspension. Again there is no sharp distinction between bed 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 sandbed 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 for development activities in the riverine environment. However, 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:

$$F = 255\ M^{-1.08} \tag{2.73}$$

where:

$$M = \frac{Sc \times W + Sb \times 2y}{W + 2y} \tag{2.74}$$

Sc = silt clay percentage in the bed

Sb = silt clay percentage in the banks

W = width of channel

y = depth of flow

Thus, M is the weighted mean percentage of fine sediments in the material composing bed and banks. Where wash load is impacted by man's activity, changes in bank stability and channel shape are possible.

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. In regions of reduced velocity and turbulence 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 load that bed material exercises its greatest influence on river form, character and resistance.

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 also 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 2-3) as an aid in estimating bed-load

Table 2-3 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)

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 (1971) as a basis for classifying alluvial channels as suspended-load, mixed-load, or bed-load channels. Table 2-4 summarizes Schumm's classification scheme which also includes both channel stabilit and the percent silt-clay, as defined in Equations (2.73 and 2.74) 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.

2.3.3.1 Bed-Load Transport. Because bed material is transported as both suspended load and bed 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 load is usually related to the tractive force or shear on the bed as in the classic Du Boys formula:

$$q_b = K \tau_o (\tau_o - \tau_c) \qquad (2.75)$$

where

Table 2-4 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)	Depositing (Excess Load)	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)

q_b = bed-load discharge per unit width of section per unit ti

K = a sediment parameter

τ_o = intensity of bed shear

τ_c = critical shear at which motion is initiated (see Figure

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

$$q_b = 10\ q\ S \frac{(\tau_o - \tau_c)}{\gamma (S_s - 1) D} \qquad (2.76)$$

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:

$$q_b = 1.606\ [3.306\ (\frac{Q_b'}{Q})(\frac{D_{90}^{1/6}}{n_b})^{3/2} (y_o S) - 0.627\ D_m]^{3/2} \qquad (2.77)$$

where

q_b = bed-load discharge 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

D_{90} = particle size at which 90 percent of the bed material is finer in mm

n_b = a roughness coefficient

y_o = depth of flow

S = energy slope

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

$$q_b = K(\tau - \tau_c)^{3/2} \tag{2.78}$$

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.

2.3.3.2 Suspended-Load Transport. The suspended bed-material discharge for steady, uniform, two-dimensional flow is:

$$q_s = \gamma \int_a^{y_o} v \; C \, dy \tag{2.79}$$

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 2-29)

y_o = depth of flow

Typical velocity and concentration profiles are shown in Figure 2-30.

To integrate Equation (2.79), v and C must be expressed as functions of y. Equation (2.70) can be used to describe the concentration profile, and a logarithmic velocity distribution (see Section 2.1.4.2) is generally used for the velocity profile:

$$\frac{v}{V_*} = 2.5 \ln 30.2 \frac{xy}{k_s} \tag{2.80}$$

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 sandbed channel)

Substitution of the velocity and concentration profiles into Equation (2.79) yields:

$$q_s = \gamma V_* C_a \int_a^{y_o} \left[\frac{a}{y_o-a} \cdot \frac{y_o-y}{y}\right]^z \left[2.5 \ln 30.2 \frac{xy}{k_s}\right] dy \tag{2.81}$$

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

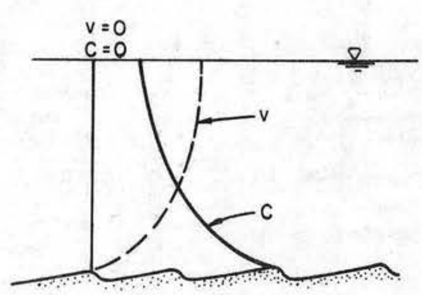


Figure 2-30 Schematic Sediment and Velocity Profiles.

2.3.3.3 Total Bed-Material Transport. Einstein's (1950) method for calculating total bed-material transport, the well known bed-load function, sums up the bed load and the suspended load. Einstein assumed that the probability of any particle of the bed 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 $\phi_* = f(\psi_*)$.

For the suspended load Einstein integrated his form of Equation (2.81). He then established a relationship between the suspended sediment load and the bed 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 bed load is given as $i_b q_b$, and the suspended load is $i_s q_s$. Thus the total bed-material discharge is

$$i_t q_t = i_s q_s + i_b q_b \tag{2.82}$$

where i_t, i_s, and i_b are the fraction of the total, suspended, and bed loads, q_t, q_s, and q_b, for a given grain size D. Using the continuous exchange of particles to relate suspended and bed loads, Equation (2.82) becomes:

$$i_t q_t = i_b q_b (1 + P_E I_1 + I_2) \tag{2.83}$$

where P_E is a transport parameter and I_1 and I_2 represent Einstein's integration of the suspended-load relationship. Equation (2.83) 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.

Colby (1964) has proposed a simple, effective method for computing bed-material discharge. Guided by Einsteins 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 2-31). With an uncorrected sediment

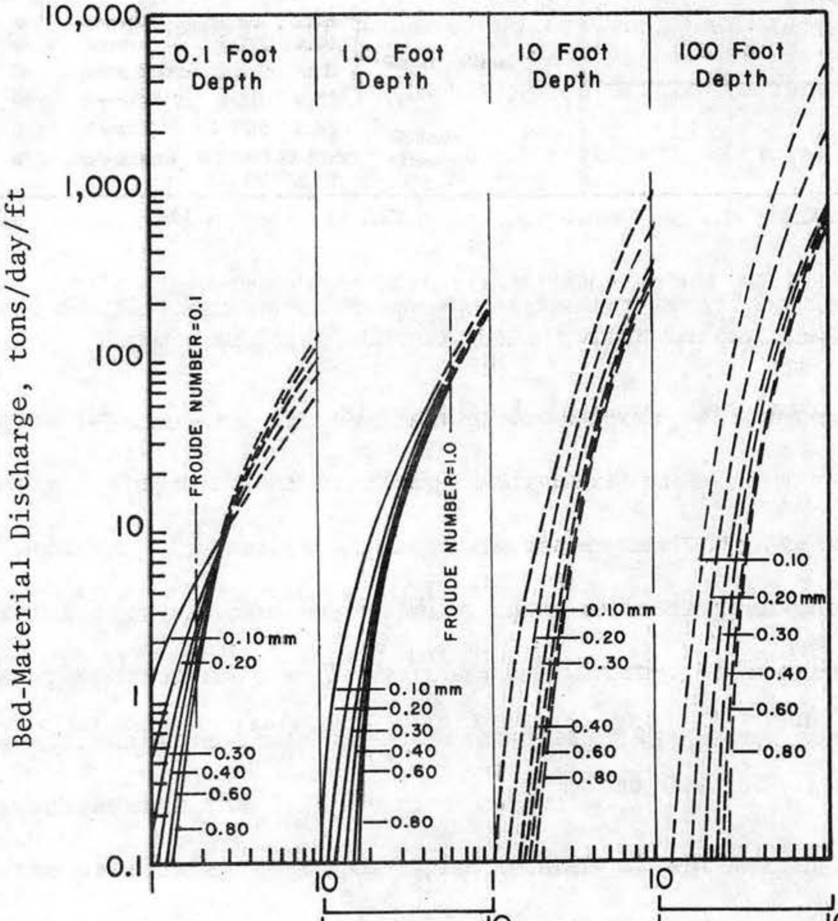


Figure 2-31 Relation of Discharge of Sands to Mean Velocity for Six Median Sizes of Bed Sands, Depth of Flow, and a Water Temperature of 60°F (after Colby, 1964).

discharge, q_n, obtained from depth, velocity, median size of the bed material (D_{50}), and Figure 2-31, the total discharge is obtained from:

$$q_t = [1 + (k_1 k_2 - 1)(0.01)k_3]q_n \tag{2.84}$$

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 2-32. 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 (2.77) can be written in the form:

$$\phi_* = \left(\frac{4}{\psi_*} - 0.188\right)^{3/2} \tag{2.85}$$

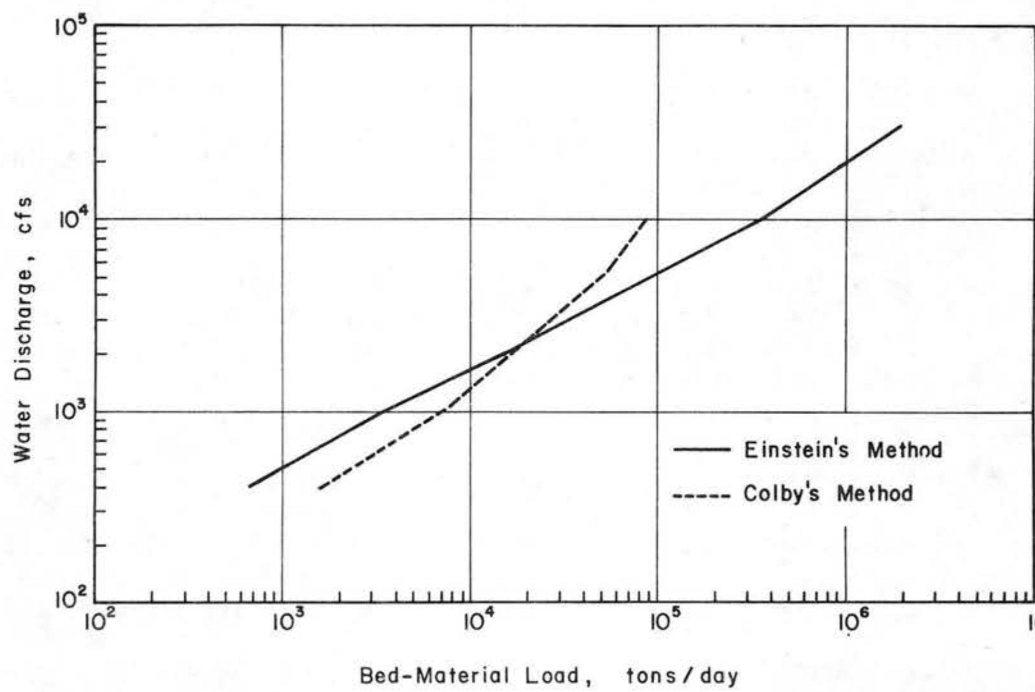


Figure 2-32 Comparison of Einstein versus Colby Methods.

Equation (2.85) is compared with Einstein's $\phi_* = f(\psi_*)$ relationship for computing bed load in Figure 2-33. The agreement is excellent.

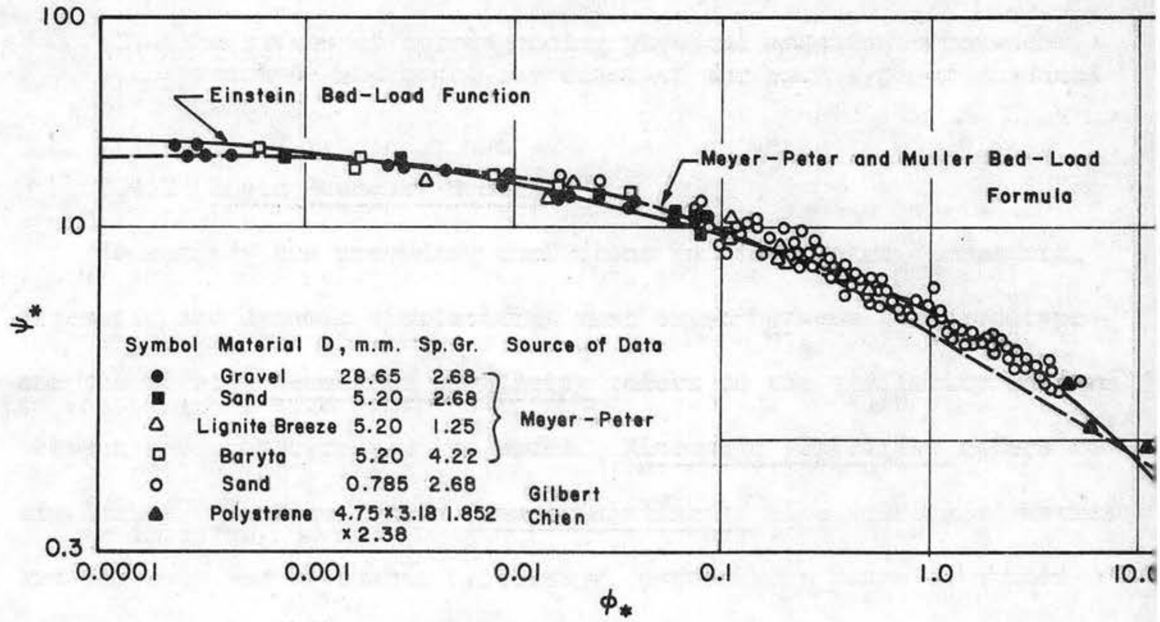


Figure 2-33 Comparison of Meyer-Peter Muller and Einstein Methods for Computing Contact Load (after Chien, 1954).

Figure 2-31 points out the strong dependence of sediment transport on 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, Gilber (1914) work indicates that for constant slope, transport capacity varie on the average with the 3.2 power of velocity.

In the preceeding sections three methods of estimating bed-materia discharge have been described, Meyer-Peter Muller, Einstein and Colby. The Meyer-Peter Muller equation is applicable to streams with little or no suspended-sediment discharge and, accordingly, is used extensively for gravel and cobble bed streams. The other two methods are used for sandbed streams. Computer programs for these methods have been develop (Mahmood and Ponce, 1975). Their use depends on the amount of informat and river characteristics.

The basic suspended-sediment equation has been presented, but methods for measuring suspended-sediment discharge have not been described. For information on these methods, reference to the publications by Guy (1969) is suggested. The data of the measured load can be used to determine the total sediment load (Colby and Hembree, 1955; Colby, 1957; Mahmood and Ponce, 1975).

2.4 Modeling of Rivers

2.4.1 Introduction

There are many problems in hydraulic engineering for which the basic equations are known but which are geometrically so complicated that the direct application of the equations is impossible. Many such problems can be solved by the use of models which duplicate this complicated geometry and in which the resulting flow patterns can be observed directly. The models may be physical models; that is small-scale physical replications. They may also be mathematical consisting of mathematical abstractions of the phenomena. Models are used to test the performance of a design or to study the details of a phenomenon. The performance tests of proposed structures can be made at moderate costs and small risks on small-scale (physical) models. Similarly, the interaction of a structure and the river environment can be studied in detail.

The natural phenomena are governed by appropriate sets of governing equations. If these equations can be integrated, the prediction of a given phenomenon in time and space domains can be made mathematically. In many cases related to river engineering, all the governing equations are not known. Also, the known equations cannot be directly treated mathematically for the geometries involved. In such cases, models are used to physically integrate the governing equations.

Similitude between a prototype and a model implies two conditions:

(1) To each point, time and process in the prototype, a uniquely coordinated point, time and process exists in the model.

(2) The ratios of corresponding physical magnitudes between prototype and model are constant for each type of physical quantity.

2.4.2 Rigid Boundary Models

To satisfy the preceding conditions in clear water, geometric, kinematic and dynamic similarities must exist between the prototype and the model. Geometric similarity refers to the similarity of form between the prototype and its model. Kinematic similarity refers to similarity of motion, while dynamic similarity is a scaling of masses and forces. For kinematic similarity, patterns or paths of motion between the model and the prototype should be geometrically similar. If similarity of flow is maintained between the model and the prototype mathematical equations of motion will be identical for the two. Considering the equations of motion, the dimensionless ratio of $\frac{V}{\sqrt{gy}}$ (Froude number) and $\frac{Vy}{\nu}$ (Reynolds number) are both significant parameters in models of rigid boundary clear water open channel flow. The Weber number $(\frac{V}{\sqrt{\sigma/\rho L}})$ which represents the ratio of inertia forces to surface tension forces can also be significant for some systems.

It is seldom possible to achieve kinematic, dynamic and geometric similarity all at the same time in a model. For instance, in open channel flow, gravitational forces predominate, and hence, the effects of the Froude number are more important than those of the Reynolds numb Therefore, the Froude criterion is used to determine the geometric scales, but only with the knowledge that some scale effects, that is, departure from strict similarity, exists in the model.

Ratios (or scales) of velocity, time, force, and other characteristics of flow for two systems are determined by equating the appropriate dimensionless number which applies to a dominant force. If the two systems are denoted by the subscript m for model and p for prototype, then the ratio of corresponding quantities in the two systems can be defined. The subscript r is used to designate the ratio of the model quantity to the prototype quantity. For example, the length ratio is given by

$$L_r = \frac{x_m}{x_p} = \frac{y_m}{y_p} = \frac{z_m}{z_p} \tag{2.86}$$

for the coordinate directions x, y, and z. Equation (2.86) assumes a condition of exact geometric similarity in all coordinate directions.

Frequently, open channel models are distorted. A model is said to be distorted if there are variables that have the same dimension but are modeled by different scale ratios. Thus, geometrically distorted models can have different scales in horizontal (x,y) and vertical (z) directions and two equations are necessary to define the length ratios in this case.

$$L_r = \frac{x_m}{x_p} = \frac{y_m}{y_p} \quad \text{and} \quad z_r = \frac{z_m}{z_p} \tag{2.87}$$

If perfect similitude is to be obtained, the relationships that must exist between the properties of the fluids used in the model and in the prototype are given in Table 2-5 for the Froude, Reynolds and Weber criteria.

In free surface flow, the length ratio is often selected arbitrarily, but with certain limitations kept in mind. The Froude number is used as a scaling criteria because gravity has a predominant effect. However, if small length ratio is used, that is, the water

Table 2-5 Scale Ratios for Similitude

		Scale Ratios		
Characteristic	Dimension	R_e	F_r	W_e
Length	L	L	L	L
Area	L^2	L^2	L^2	L^2
Volume	L^3	L^3	L^3	L^3
Time	T	$\rho L^2/\mu$	$(L\rho/\gamma)^{1/2}$	$(L^3\rho/\sigma$
Velocity	L/T	$\mu/L\rho$	$(L\gamma/\rho)^{1/2}$	$(\sigma/L\rho)$
Acceleration	L/T^2	μ^2/ρ^2L^3	γ/ρ	σ/L
Discharge	L^3/T	$L\mu/\rho$	$L^{3/2}\,\frac{\gamma}{\rho}^{1/2}$	$L^{3/2}(\sigma/\rho$
Mass	M	$L^3\rho$	$L^3\rho$	$L^3\rho$
Force	ML/T^2	μ^2/ρ	$L^3\gamma$	$L\sigma$
Density	M/L^3	ρ	ρ	ρ
Specific weight	M/L^2T^2	$\mu^2/L^3\rho$	γ	σ/L^2
Pressure	M/LT^2	$\mu^2/L^2\rho$	γ	σ/L
Impulse and momentum	ML/T	$L^2\mu$	$L^{7/2}(\rho\gamma)^{1/2}$	$L^{5/2}(\rho\sigma)$
Energy and work	ML^2/T^3	$L\mu^2/\rho$	$L^4\gamma$	$L^2\sigma$
Power	ML^2/T^3	$\mu^3/L\rho^2$	$\frac{L^{7/2}\gamma^{3/2}}{\rho^{1/2}}$	$\sigma^{3/2}(L/\rho$

depths are very shallow, then surface tension forces, effects of which are included in the Weber number, may become important and complicate the interpretations of results of the model. The length scale is made as large as possible so that the Reynolds number is sufficiently large that friction becomes a function of the boundary roughness and essentially independent of the Reynolds number. A large length scale also insures that the flow is turbulent in the model as it usually is in the prototype.

The boundary roughness is characterized by Manning's roughness coefficient, n, in free surface flow. Analysis of Manning's equation and substitution of the appropriate length ratios, based upon the Froude criterion, results in an expression for the ratio of the roughness which is given by

$$n_r = L_r^{1/6} \tag{2.88}$$

It is not always possible to achieve boundary roughness in a model and prototype that correspond to that required by Equation (2.88) and additional measures, such as adjustment of the slope, may be necessary to offset disproportionately high resistance in the model.

2.4.3 Mobile Bed Models

In modeling response to development works in the river environment, three-dimensional mobile bed models are often used. These models have the bed and sides molded of materials that can be moved by the model flows. Similitude in mobile bed models implies that the model reproduces such fluvial processes as bed scour, bed deposition, lateral channel migration, and varying boundary roughness. It has not been considered possible to faithfully simulate all of these processes simultaneously on scale models. Distortions of various parameters are often made in such models.

Two approaches are available to design mobile bed models. One is the analytical derivation of distortions explained by Einstein and

Chien (1956) and the other is based on hydraulic geometry relationships given by Lacey, Blench, and others (Mahmood and Shen, 1971). In both of these approaches, a first approximation of the model scales and distortions can be obtained by numerical computations. The model is built to these scales and then verified for past information obtained from the prototype. In general, the model scales need adjusting during the verification stage.

The model verification consists of the reproduction of observed prototype behavior under given conditions on the model. This is specifically directed to one or more alluvial processes of interest. For example, a model may be verified for bed-level changes over a certain reach of the river. The predictive use of the model should be restricted to the aspects for which the model has been verified. This use is based on the premise that if the model has successfully reproduced the phenomenon of interest over a given hydrograph as observed on the prototype, it will also reproduce the future response of the river over a similar range of conditions.

Mobile bed models are more difficult to design and their theory is extremely complicated as compared to clear water rigid bed models. However, many successful examples of their use are available. In general, all important river training and control works are studied on physical models. The interpretation of results from a mobile bed model requires a basic understanding of the fluvial process and some experience with such models. In many cases, where it is possible to obtain only qualitative information from mobile bed models this information is of great help in comparing the performance of different designs.

2.4.4 Mathematical Models

A physical scale model is a means for extracting information from some source other than from the prototype. As Gessler (1971) points out, a physical model can be looked on as an analog computer, since there is a high degree of analogy between prototype and model. With a distorted physical model the geometrical analogy is weakened considerably, but still, under most conditions, the analogy of the overall behavior is strong.

Once one gets used to the idea of looking at a model as an analog computer the next logical step would be to model the process under study on a digital computer in numerical form. This of course requires that a "complete" set of governing equations (some of them differential equations) is available. Such equations would include basic flow equations, the differential equation of non-uniform and unsteady flow, the sediment transport equation, the differential equation formulating continuity of sediment transport, and criterion to predict the bed deformations, just to mention the most obvious equations involved. It is clear that the interaction between these equations is extremely complex. This, after all, is the reason for attempting to model these processes physically. But with the availability of high speed digital computers it becomes entirely feasible to study some of the characteristics of a river system numerically. Of course the results cannot be better than the basic equations used in the analysis, and most equations available are for one- or two-dimensional flow fields only. But when an overall river system is considered, a river can be viewed as a highly two-dimensional system, and with certain simplifying assumptions a river can be modeled as a one-dimensional system. It is only when one starts looking into the details that three-dimensional processes become important (Gessler, 1971).

A one-dimensional model for water and sediment routing in natural channels has been developed at Colorado State University (Chen, 1973). This model has been applied to a variety of problems involving the evaluation or prediction of river response to development works. This model constitutes an excellent example of the application of numerical modeling techniques to river response problems, and accordingly, is described briefly in the following paragraphs.

The model for water and sediment routing is developed by describin 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 one-dimensional differential equations of gradually varied, unsteady flow in natural alluvial channels can be derived base 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 dat

The three basic equations thus derived are:

1. The sediment continuity equation

$$\frac{\partial Q_s}{\partial x} + p\,\frac{\partial A_d}{\partial t} + \frac{\partial AC_s}{\partial t} - q_s = 0 \tag{2.89}$$

2. The flow continuity equation

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + \frac{\partial A_d}{\partial t} - q_\ell = 0 \tag{2.90}$$

3. The flow momentum equation

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

or

$$\frac{\partial \rho Q}{\partial t} + V\,\frac{\partial \beta\rho Q}{\partial x} + \beta\rho V\,\frac{\partial Q}{\partial x} - \beta\rho V^2 T\,\frac{\partial y}{\partial x} + gA\,\frac{\partial \rho y}{\partial x} = \rho gA(S_o - S_f + D_\ell) + \beta\rho V^2 A_x^y \tag{2.91}$$

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 2-34).

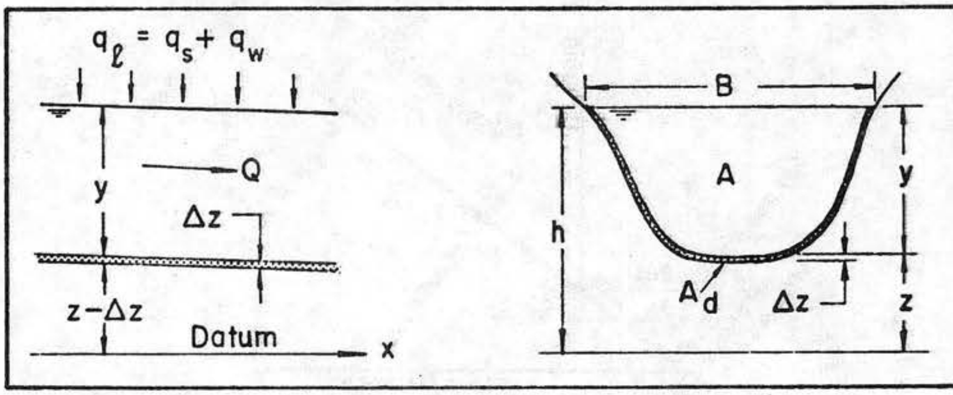


Figure 2-34 Definition Sketch of an Alluvial Channel (after Simons et al., 1975).

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

$$S_o = -\partial z/\partial x \tag{2.92}$$

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:

$$q_{\ell 1} = -\frac{A_f}{\Delta x \Delta t} \Delta h \tag{2.93}$$

where A_f is the surface area of the floodplain, and Δx is the length of the floodplain along the main channel. Equation (2.93) is based on the assumptions that the transverse water surface (normal to the main flow direction) is horizontal and longitudinal flow on the floodplain is negligible. Accordingly, the lateral sediment inflow q_s has its natural and man-induced components, q_{s1} and q_{s2}, in which

$$q_{s1} = q_{\ell 1} C_b \tag{2.94}$$

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.

When the model is applied to a specific problem such as the modeling of river response and geomorphic change in the navigation pools of the Upper Mississippi River navigation system, additional relationships are required to simulate the effects of structures and tributaries in the modeled reach. For example, in a model of the Pool 24, 25, and 26 reach of the Upper Mississippi River (Simons et al., 1975), the effects of locks and dams are accounted for by applying the continuity equations for both sediment and water discharge across each lock and dam, supplemented by a gate discharge equation as shown in Figure 2-35 (compare Figure 2-35 with the prototype reach shown in Figure 5-12). The interaction between the Upper Mississippi and each of its major tributaries in this reach (the Illinois and Missouri Rivers) is simulated by continuity and energy relationships written at each

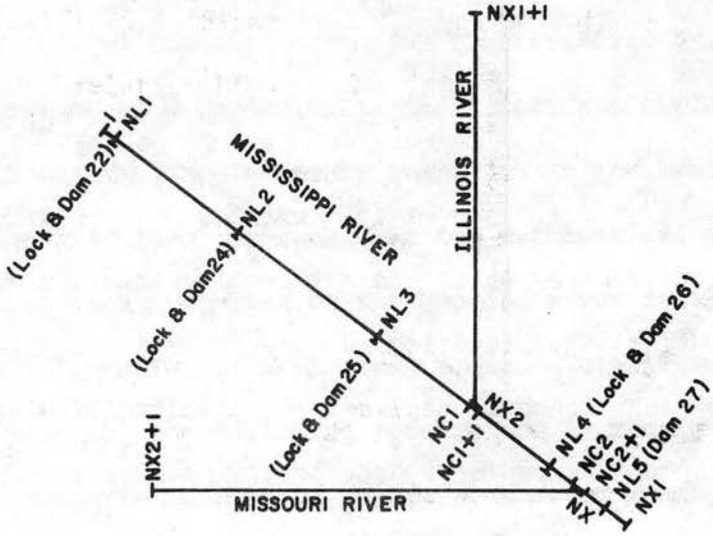


Figure 2-35 Schematic Diagram of the Study River Reach (after Simons et al., 1975).

tributary confluence (Figure 2-35). The set of basic equations and supplementary relationships, Equations (2.89-2.94), plus the equations written across each lock and dam and tributary confluence govern the water flow and sediment movement in the 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 space lines x_i and x_{i+1} with the time lines t^j and t^{j+1} are shown by Figure 2-36. These are

$$f \approx \frac{1}{2}\left(f_i^j + f_{i+1}^j\right) \tag{2.95}$$

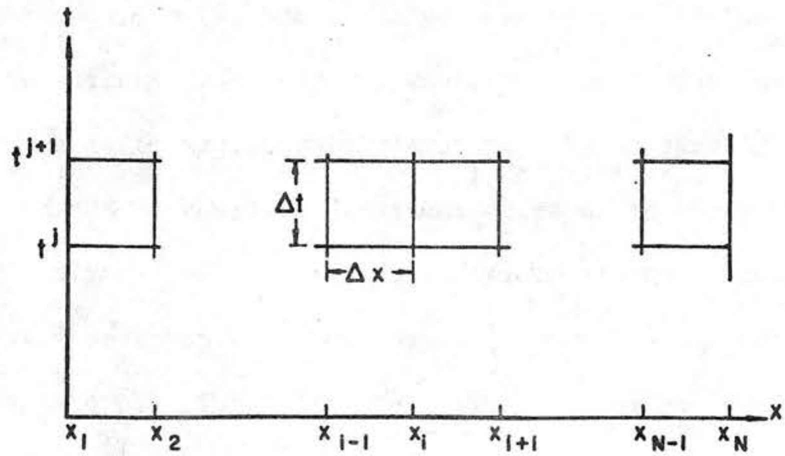


Figure 2-36 Network for the Implicit Method.

$$\frac{\partial f}{\partial x} \approx \frac{1}{\Delta x} (f_{i+1}^{j+1} - f_i^{j+1}) \tag{2.96}$$

and

$$\frac{\partial f}{\partial t} \approx \frac{1}{2\Delta t} [(f_i^{j+1} - f_i^j) + (f_{i+1}^{j+1} - f_{i+1}^j)] \tag{2.97}$$

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 (2.95), (2.96), and (2.97) 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 materia

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

For the specific problem of modeling the lower three pools of the Upper Mississippi River navigation system, the study reach (Figure 2-3 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 utilizi 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 2-37. Good 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.

The results of applying this mathematical model to the evaluation and prediction of river response in the Pool 24, 25, and 26 reach of the Upper Mississippi are described in detail in Chapter 5, and the use of the model to evaluate alternatives to current dredging and disposal pratices is discussed in Chapter 7. Data requirements for mathematical modeling are included in Chapter 8.

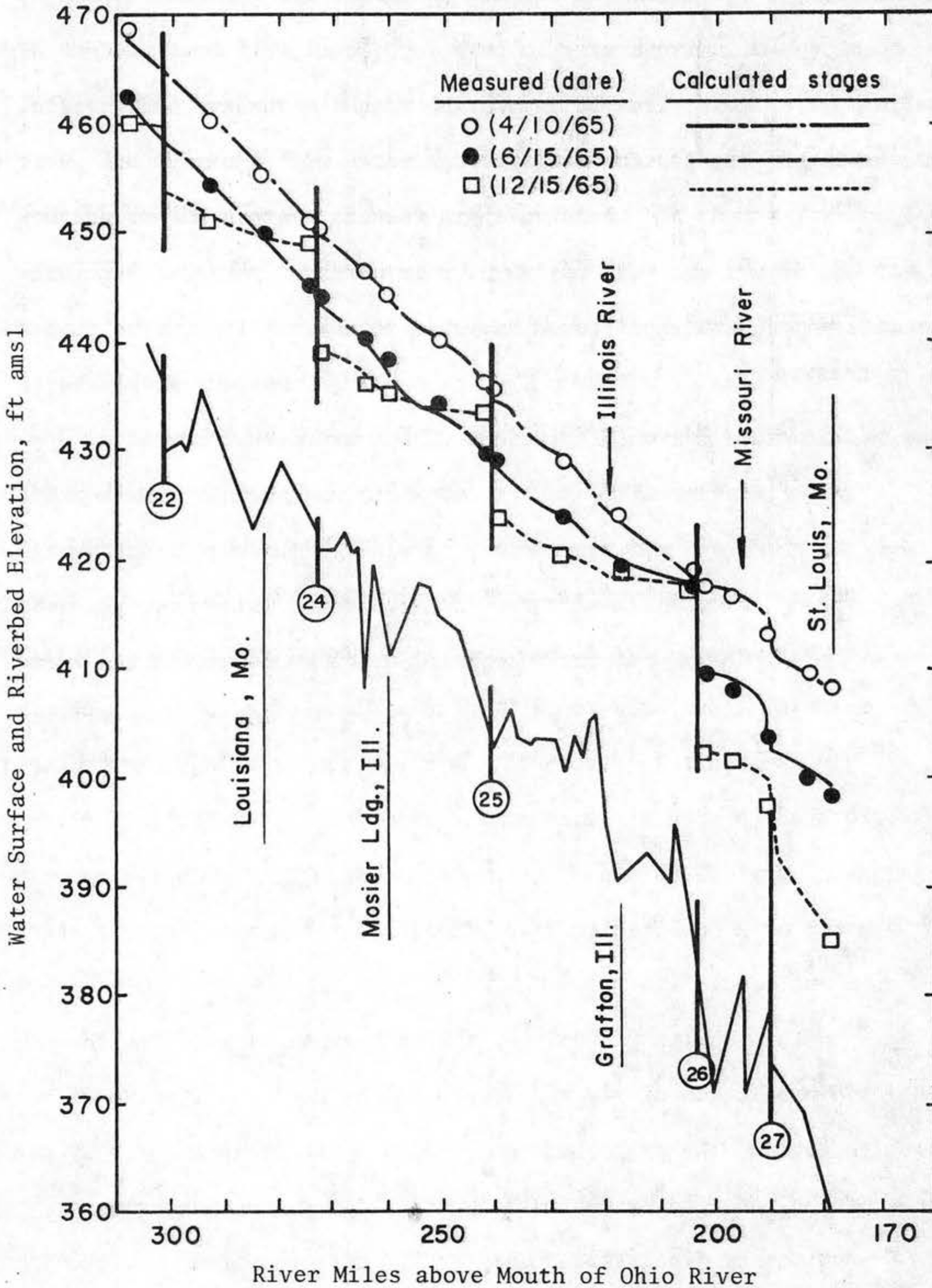


Figure 2-37 Mathematical Reproduction of 1965 Water Surface Profile i the Upper Mississippi River (after Simons et al., 1975).

The application of mathematical modeling is not limited to the main channel of a river system. The management of watersheds and river basins requires, in general, a complete knowledge of the interrelations between ecology and environment. The watershed response to developments, either natural or man-induced, must be anticipated correctly if progress is to be made towards wise use of our nation's natural resources.

The physical quantities which describe the major watershed response are the water hydrograph and yield, the sediment hydrograph and yield, and the resultant watershed stream morphology. Because the physical processes governing watershed behavior are very complicated many past studies have utilized a statistical interpretation of observed watershed response data. The Unit Hydrograph Method for water routing and the Universal Soil-loss Equation for estimating soil erosion are examples of these types of statistical studies. However, it is difficult to predict the response of a watershed to various land developments or treatments using these methods because they are based on the assumption of homogeneity in both time and space. Numerical modeling using equations describing the physical processes provides a viable method of estimating time-dependent watershed response. In recognition of the necessity of such a model, Colorado State University has developed a numerical computer program employing the formulation of the basic physical processes to determine water and sediment hydrographs and yields from small watersheds.

The Colorado State University (CSU) watershed model simulates the land surface hydrologic cycle, sediment production, and water and sediment movement on small watersheds. Conceptually the watershed is divided into an overland flow part and a channel system part. Different physical processes are important for the two different environments. In the overland flow loop, processes of interception, evaporation, infiltration, raindrop impact detachment of soil, erosion by overland flow, and overland flow water (surface and subsurface) and sediment routing to the nearest channel are simulated. In channel system loop, water and sediment contributed by overland flow are routed and the amount of channel erosion or sediment deposition through the channel system is determined.

In the CSU watershed model, emphasis is on the mechanics of water and sediment routing and the model is set up for single storm hydrograph computations. As yet, no attempt has been made to simulate the long-term water balance in the watershed. However, any valid water balance model, such as Stanford Watershed Model IV, can be interfaced with the CSU model.

Chapter 3

RIVER MORPHOLOGY

3.1 Concepts of Fluvial Geomorphology

3.1.1 Introduction

Fluvial geomorphology is the cumbersome title applied to the geologic speciality that treats the morphology of rivers and river systems. As Schumm (1971) notes in a review on the concepts of fluvial geomorphology, it is more than the form, behavior, and mechanics of rivers that interests the geomorphologist. As geology is concerned with the history of Earth through billions of years, the geomorphologist views the fluvial landscape in a historical perspective. He utilizes the concept of unlimited time during which the landscape evolved to its present configuration. The engineer may question the relevance of the historical approach to river mechanics, but its contribution should not be ignored. The investigation of river response to the climatic changes and diastrophic events of Earth's evolution provides information and insight into long-term river adjustment to altered hydrologic conditions.

Geologists, of course, are vitally concerned with modern river morphology and mechanics, because this information makes possible the interpretation of ancient river deposits. These interpretations are of considerable economic importance. For example, petroleum is now being found in sediments of river systems of late Paleozoic and Mesozoic ages (about 250 to 70 million years ago) in the United States and Canada. In most cases, these are valley-fill deposits composed of a complex of channel deposits rather than a single channel. Uranium deposits of the Colorado Plateau are located in paleochannels that were functioning about 100 million years ago. In addition, the rich gold deposits of the Witwatersrand basin of South Africa are ancient placers (older than 1/2 billion years), complex river deposits laid down on an alluvial plain. This gold is being mined at great depths, and any improvement of our understanding of the depositional environment of the Witwatersrand sediments could significantly reduce the costs involved in exploiting these mineral deposits.

Much of the geomorphic effort is descriptive in order to establish the chronology of events in the history of a river system. Nevertheless the insight gained into the long-term response of rivers to major climatic change or to movements of Earth's crust has bearing on problems of modern river regulation and control (Schumm, 1971).

3.1.2 Genetic Classification of Streams

Various methods are used to classify rivers according to their age. One of the methods used by geomorphologists, and widely accepted by the engineering profession, classifies streams as youthful, mature, and old. Youthful implies the initial state of streams. As channels are first developed in the earth's surface by the flowing water, they are generally V-shaped, very irregular and consist of fractured erosive and nonerosive materials. Examples of youthful streams are mountain streams and their tributaries developed by overland flow.

There is no clean line between youthful and mature rivers. In the case of mature channels, the river valleys have widened, the river slopes are flat, and bank cutting has largely replaced downward cutting. The streambed has achieved a graded condition, that is the slope and the energy of the stream are just sufficient to transport the material delivered to it. With mature channels, narrow floodplains and meanders have formed. The valley bottoms are sufficiently wide to accommodate agricultural and urban developments, and where development has occurred usually channel stabilization works and other improvements have been made to prevent lateral migration of the river.

River channels classified as old are extensions in age of the mature channel; as erosion continues, the river valleys develop so that their characteristics include greater width, low relief, the stream gradient has flattened further, and meanders and meander belts that have developed are not as wide as the river valley. Natural levees have formed along the stream banks. Landward of the natural levees, there are swamps. The tributaries to the main channel parallel the main channel sometimes for long distances before there is a breach in the natural levee that permits a confluence. In conjunction with an old river and its river valley, wide areas are available for cultivation, improvements of all types are built, and flood levees are generally required to protect those occupying the valley. Because of the more sophisticated development of the river valley, channel stabilization and contraction work such as revetments and dikes are generally present.

It should be emphasized that the preceding concept of the fluvial cycle is not accepted by all geologists. For example, some consider a channel to be mature only after the trunk stream as well as the side streams have achieved a graded condition. Some define old age as a condition when the entire river system is graded. Graded streams are referred to as those that have achieved slopes such that their energy is just sufficient to transport the material through the system that is delivered to the streams. This concept can only be applied as an average condition extending over a period of years. No stream is continuously graded. A poised stream refers to one that neither aggrade or degrades its channel over time. Both graded and poised streams are delicately balanced. Any change imposed on the river system will alter the balance and lead to actions by the stream to reestablish balance. For example, a graded or poised stream may be subjected locally to the development of a cutoff. The development of the cutoff increases the channel slope, increases velocity, and increases transport at least locally. Changes in these variables cause changes in the channel and deposition downstream. The locally steepened slope gradually extends itself upstream attempting to reestablish equilibrium.

3.1.3 Floodplain and Delta Formations

Over time, the highlands of an area are worn down. The streams erode their banks. The material that is eroded is utilized further downstream to build banks and to further the meandering process. Stream move laterally pushing the highlands back. Low flat valley land and floodplains are formed. As the streams transport sediment to areas of flatter slopes, and in particular to bodies of water where the velocity and turbulence are too small to sustain the transport of the material, the material is deposited forming deltas. As deltas build outward the upriver portion of the channel is elevated through deposition and become part of the floodplain. Also, the stream channel is lengthened and the slope is further reduced. The upstream riverbed is filled in and averag flood elevations are increased. As it works across the river valley this type of development causes the total floodplain to raise in elevation. Hence, even old streams are far from static. Old rivers meander, are affected by changes in sea level, are influenced by movements of the earth's crust, are changed by delta formations or glaciation, and are subject to modifications due to climatological changes and as a consequence of man's development of them.

3.1.4 Alluvial Fans

Another feature of rivers is alluvial fans. They occur wherever there is a change from a steep to a flat gradient. As the bed material and water reaches the flatter section of the stream, the coarser bed material can no longer be transported because of the sudden reduction in both slope and velocity. Consequently, a cone or fan builds out as the material is dropped. The steep side of the fan faces the floodplain. There is considerable similarity between a delta and an alluvial fan. Both result from reductions in slope and velocity. Both have steep slopes at their outer edges. Both tend to reduce upstream slopes. Alluvial fans like deltas are also characterized by unstable channel geometries and rapid lateral movement. A feature very similar to the delta, develops where a steep tributary enters a main channel. The steep channel tends to drop part of its sediment load in the main channel building out into the main stream. In some instances, the main stream can be forced to make drastic changes at the time of major floods by the stream's tributaries.

3.2 Stream Form and Classification

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 3-1).

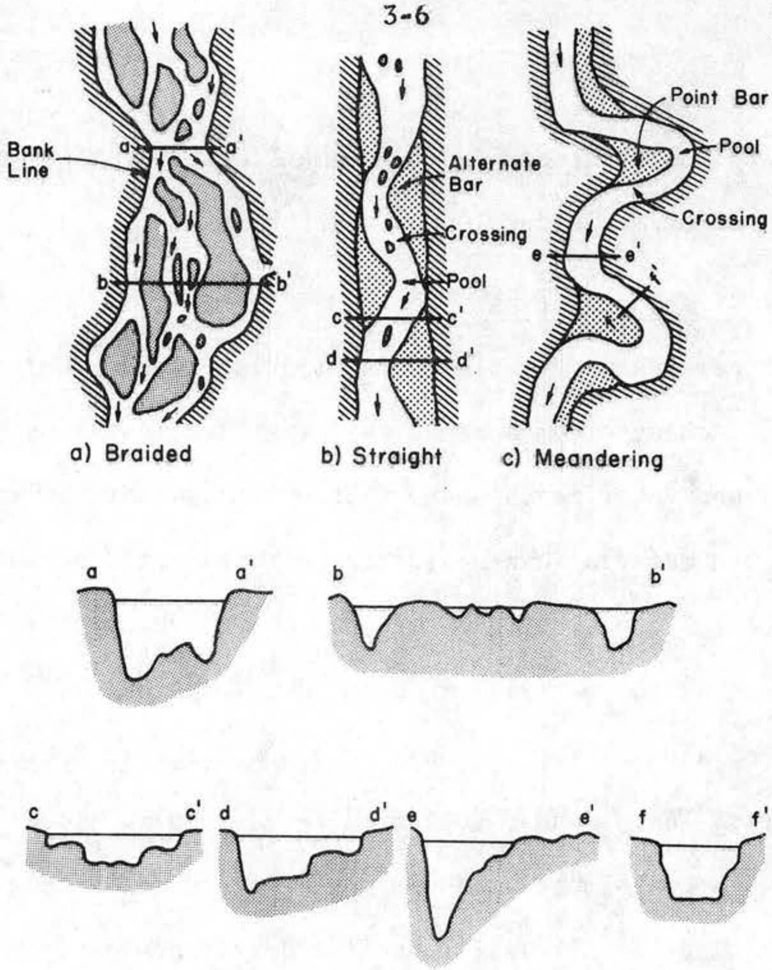


Figure 3-1 River Channel Patterns.

3.2.1 Straight Channels

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 3-1b). 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 3-1b) 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 3-2). 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.

3.2.2 The Braided Stream

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 3-1a). 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

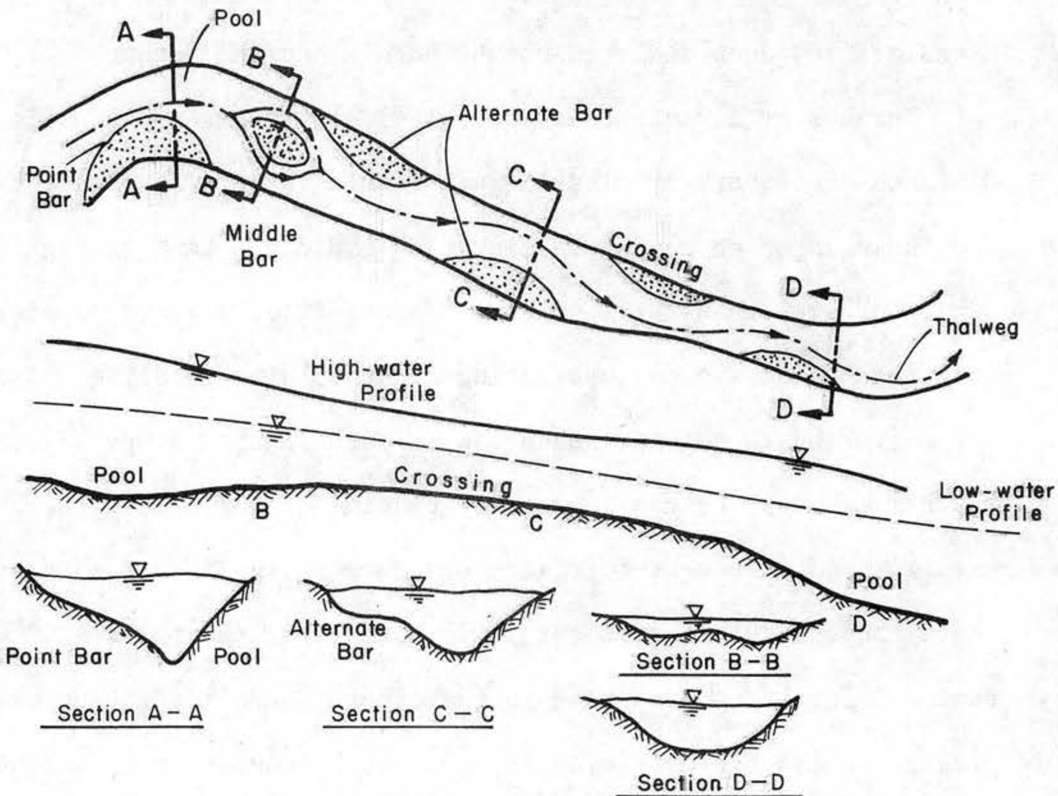


Figure 3-2 Plan View and Cross Section of a Meandering Stream.

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.

Another cause of braiding is easily eroded banks. If the banks are easily eroded, the stream widens at high flow and at low flow bars form which become stabilized, forming islands. In general, then, a braided channel has a steep slope, a large bed-material load in comparison with its suspended load, and relatively small amounts of silts and clay in the bed and banks. Figure 3-3 summarizes the various conditions for multiple channel streams. The braided stream is difficult to work with in that it is unstable, changes its alignment rapidly, carries large quantities of sediment, is very wide and shallow even at flood flow and is in general unpredictable.

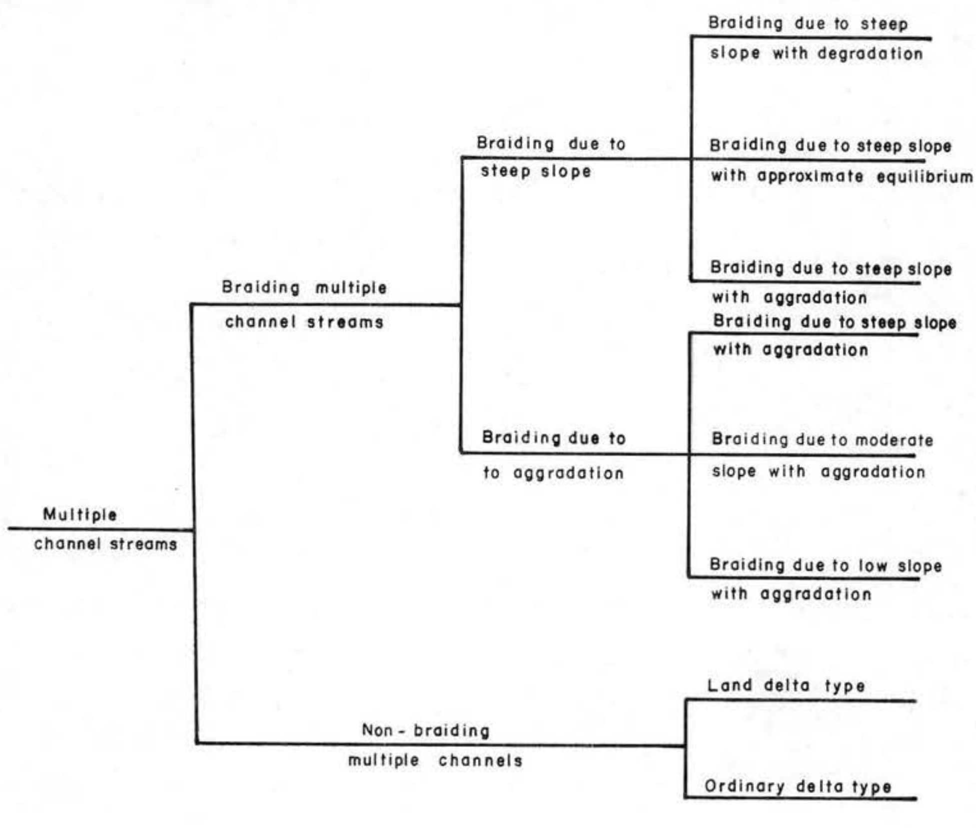


Figure 3-3 Types of Multi-Channel Streams.

3.2.3 The Meandering Channel

A meandering channel is one that consists of alternating bends, giving an S-shape appearance to the plan view of the river (Figure 3-1c 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 thalwe flows from a pool through a crossing to the next pool forming the typical S-curve of a single meander loop. Figure 3-4 shows the classic Greenville meanders on the Lower Mississippi River.

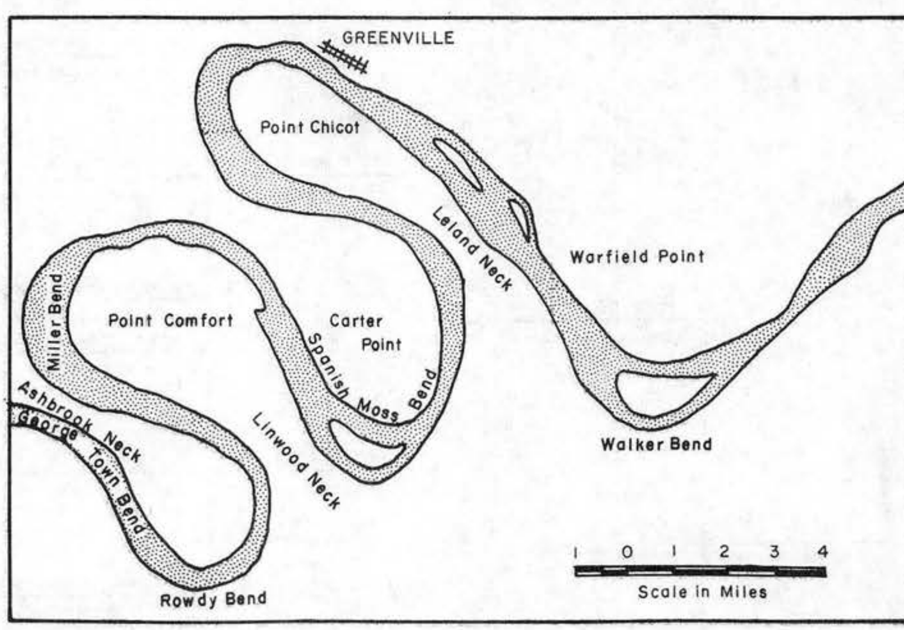


Figure 3-4 Meanders in Mississippi River near Greenville, Mississippi.

As shown schematically in Figure 3-1, the pools tend to be somewha 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. At low flows the local slope is steeper and velocities are larger in the crossing than in the pool. At low stages the thalweg is located very close to the outside of the bend. At higher stages, the thalweg tends to straighten. More specifically the thalweg moves away from the outside of the bend encroaching on the point bar to some degree. In the extreme case, the shifting of the current causes chute channels to develop across the point bar at high stages. Figure 3-2 shows the plan view and cross section of a typical meandering stream. In this figure, one can observe the position of the thalweg, the location of the point bars, alternate bars and the location of the pools and crossings. Note that in the crossing the channel is shallow compared to pools and the banks may be more subject to erosion.

3.2.4 The Meandering Process

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; however, most arguments invoke the existence of secondary currents in the bendway as a prime contributor to the meandering process.

Alluvial channels of all types deviate from a straight alignment. The thalweg oscillates transversely and initiates the formation of bends. In general, the river engineer concerned with channel stabilization should not attempt to develop straight channels. In a straight channel the alternate bars and the thalweg (the line of greatest depths along the channel) are continually changing; thus the current is not uniformly distributed through the cross section but is deflected toward one bank and then the other. Sloughing of the banks, nonuniform deposition of bed load by debris such as trees, and the Coriolis force have been cited as causes for meandering of streams. When the current is directed toward a bank, the bank is eroded in the area of impingement and the current is deflected and impinges upon the opposite bank further downstream. The angle of deflection of the thalweg is affected by the curvature formed in the eroding bank and the lateral depth of erosion.

In general, bends are formed by the process of erosion and deposition. Erosion without deposition to assist in bend formation would result only in escalloped banks. Under these conditions the channel would simply widen until it was so large that the erosion would terminate. The material eroded from the bank is normally deposited over a period of time on the point bars that are formed downstream. The point bars constrict the bend and enable erosion in the bend to continue accounting for the lateral and longitudinal migration of the meandering stream. Erosion is greatest across the channel from the point bar. As the point bars build out from the downstream sides of the points, the bends gradually migrate down the valley. The point bars formed in the bendways clearly define the direction of flow. The bar generally is streamlined and its largest portion is oriented downstream. If there is very rapid caving in the bendways upstream the sediment load may be sufficiently large to cause middle bars to form in the crossing.

As a meandering river system moves laterally and longitudinally, the meander loops move at an unequal rate because of the unequal erodibility of the banks. This causes a tip or bulb to form and ultimately this tip or bulb is cut off. After the cutoff has formed, a new bend may slowly develop. Its geometry depends upon the local slope, the bank material, and the geometry of the adjacent bends. Over time the local steep slope caused by the cutoff is distributed both upstream and downstream. Years may be required before a configuration characteristic of average conditions in the river is attained.

When a cutoff occurs, an oxbow lake is formed. Oxbow lakes may persist for long periods of time before filling. Usually the upstream end of the lake fills quickly to bank height. Overflow during floods carries fine materials into the oxbow lake area. The lower end of the oxbow remains open and the drainage and overland flow entering the system can flow out from the lower end. The oxbow gradually fills with fine silts and clays. Fine material that ultimately fills the bendway is plastic and cohesive. As the river channel meanders it encounters old bendways filled with cohesive materials (referred to as clay plugs). These plugs are sufficiently resistant to erosion to serve as essentially semipermanent geologic controls. Clay plugs can drastically affect river geometry. The development of cutoffs and oxbow lakes is shown schematically in Figure 3-5.

The variability of bank materials and the fact that the river encounters such features as clay plugs causes a wide variety of river forms coincident with a meandering river. The meander belt formed by a meandering river is often fifteen to twenty times the channel width.

3.2.5 Natural Levees and Back Swamps

Natural levees are a characteristic of old river systems. The natural levees near the river are rather steep because coarse material drops out quickly. Farther from the river the gradients are flatter

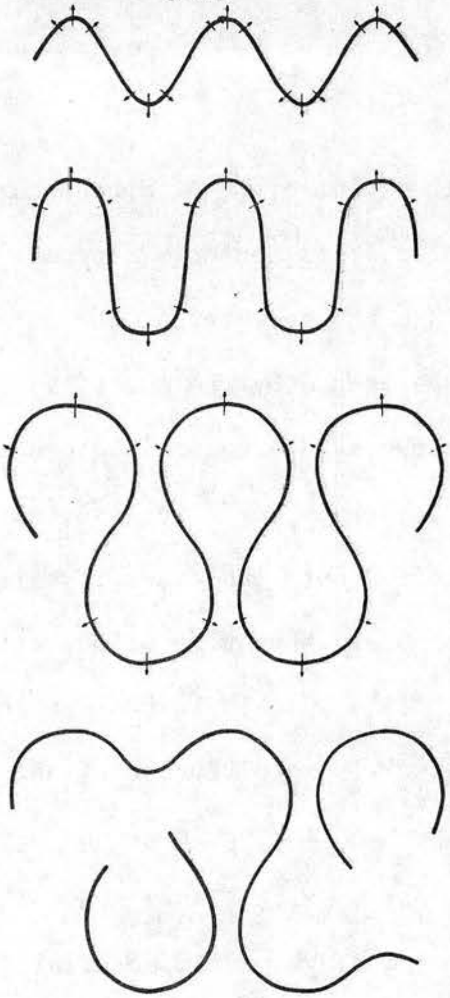


Figure 3-5 Development of Natural Cutoffs (after NEDECO, 1959).

and the finer materials drop out. Beyond the levees are the swamp areas. On the lower Mississippi River, natural levees on the order of ten feet in height are common. The rate of growth of natural levees is smaller after they reach a height equal to the average annual flood stage.

3.2.6 The Continuum of Channel Patterns

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 3-6). 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 3-6. This

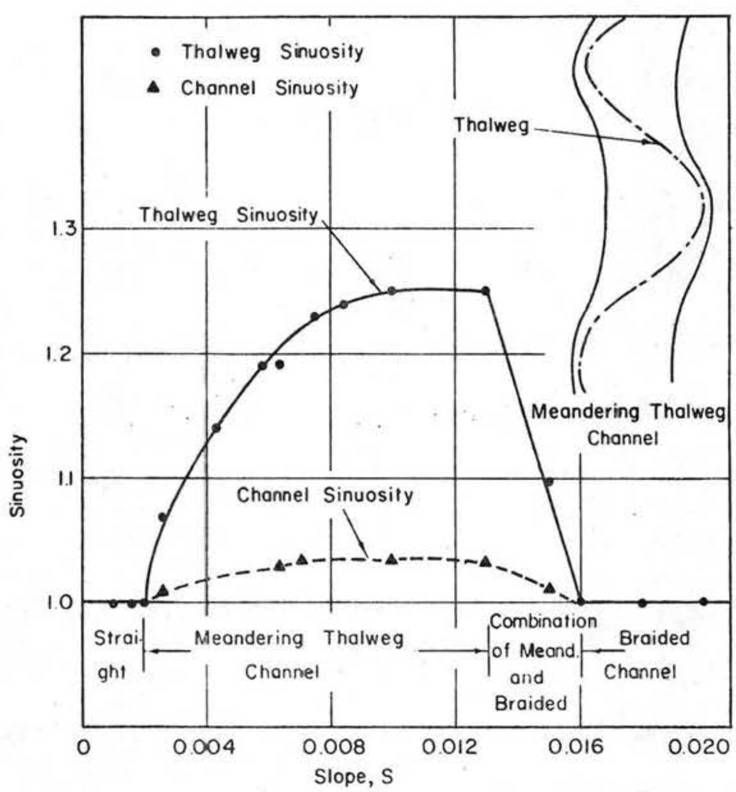


Figure 3-6 Sinuosity versus Slope for a Constant Discharge of 0.15 cfs (after Khan, 1971).

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

$$SQ^{1/4} = K \tag{3.1}$$

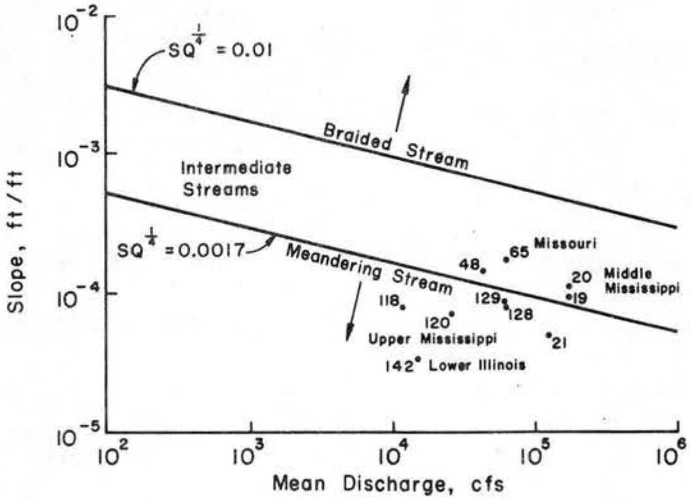
fits a large amount of data from meandering sand streams. Here, S is the channel slope, Q is the water discharge, and K is a constant. Figure 3-7 summarizes Lane's plots and shows that when

$$SQ^{1/4} \leq 0.0017 \qquad (3.2$$

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

$$SQ^{1/4} \geq 0.01 \qquad (3.3$$

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.



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 - LaCrosse to Lansing
128 Upper Mississippi - Hannibal to Louisiana
129 Upper Mississippi - Louisiana to Grafton
142 Lower Illinois River

Figure 3-7 Slope-Discharge Relation for Braiding or Meandering in Sand Bed Streams (after Lane, 1957).

With reference to Figure 3-7, 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 are shown on Figure 3-7. 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) recognized the braided character of some reaches in the Upper Mississippi and noted 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.

These conditions and the resulting channel pattern are discussed in detail in Chapter 4.

3.2.7 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. 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:

$$S_x = S_o \, e^{-\alpha x} \tag{3.4}$$

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 3-8). 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

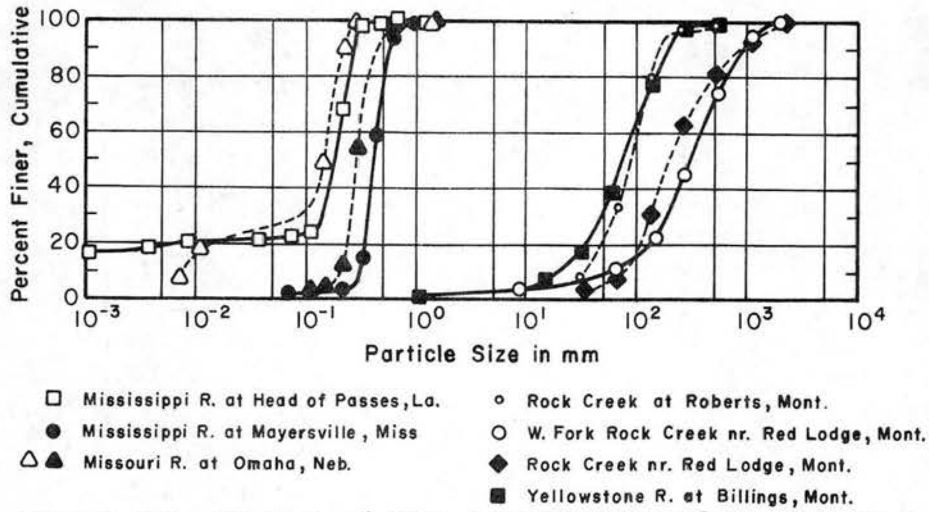


Figure 3-8 Samples of Size Distribution of Bed Material, Yellowstone-Missouri-Mississippi River System (after Leopold et al., 1964).

shape, roughness, and specific gravity, the end result of these processes is the observed reduction of bed material size along the direction of transport. The change in particle size with distance downstream can be expressed as

$$D_{50x} = D_{50o}\, e^{-\beta x} \qquad (3.5)$$

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 3-9.

The longitudinal profile of an alluvial river is not static. It adjust to continually changed input conditions of water and sediment discharge. Although adjustment to input conditions changes is realized by modification of channel geometry, roughness, and other parameters including gradient, a simplified model of the adjustment

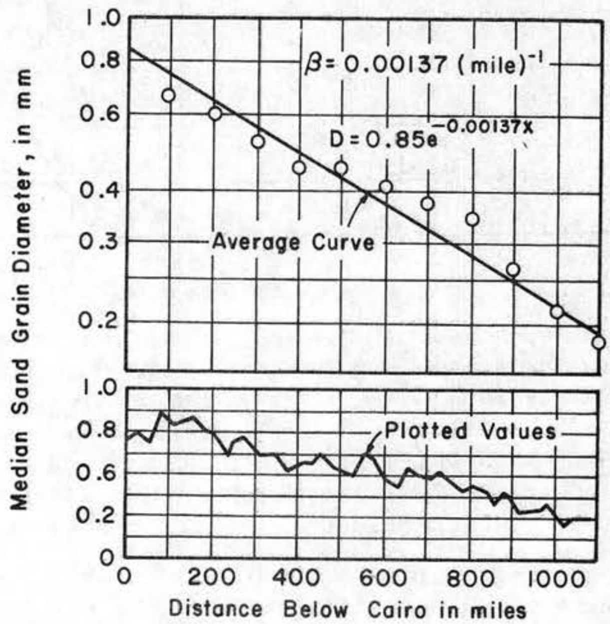


Figure 3-9 Particle Size Reduction along the Mississippi River.

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 build up the channel bed, causing an increased slope below the point and thus an increased ability to transport (Figure 3-10a). 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, the flow will scour its channel at the point of excess capacity (Figure 3-10b). 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.

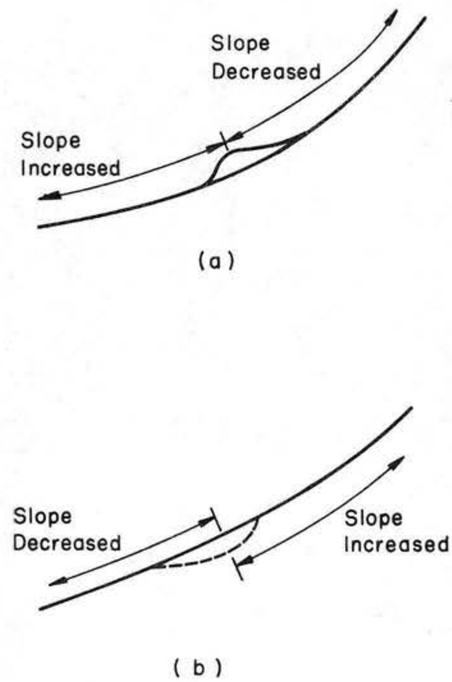
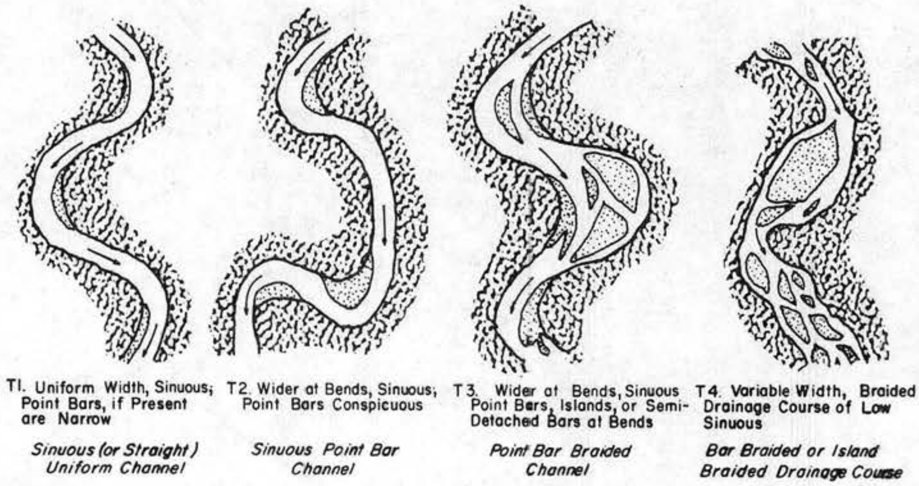


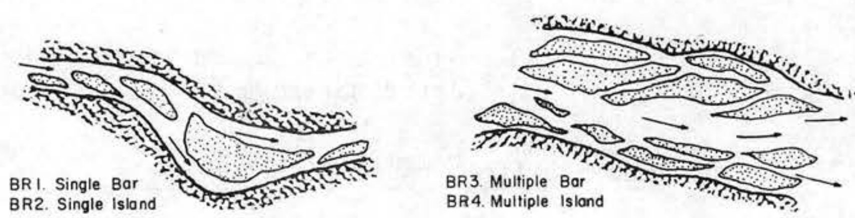
Figure 3-10 Adjustments in Longitudinal Profile (after Morisawa, 1968).

3.2.8 Subclassification of River Channels

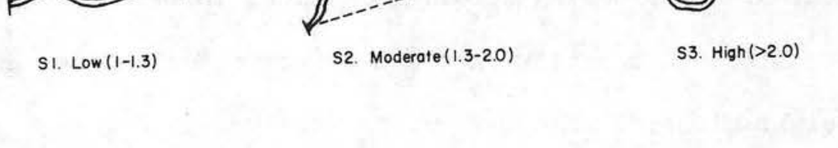
There are subclassifications within the major types of meandering, straight and braided channels that are of use to the geomorphologist and engineer. Low, moderate and high sinuosity are illustrated in Figure 3-11c. Classification based on oxbow lakes is illustrated in Figure 3-11d. In Figure 3-11e types of meander scroll formations are illustrated. By studying scroll formations in terms of age of vegetation it is possible to quantify rate and direction of channel migration. The bank height classification of rivers is given in Figure 3-11f. Bank height is often an important index to age and activity of the river. Classification based on natural levees is illustrated in Figure 3-11g. As pointed out earlier, well-developed levees are associated with older rivers. Typical modern floodplains are illustrated in Figure 3-11h. The floodplain that is broad in relation to the channel width is indicative of an older river. Conversely when the river valley is



(a) Variability of unvegetated channel width: channel pattern at normal discharge

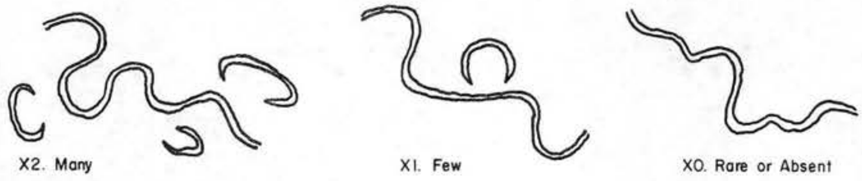


(b) Braiding patterns

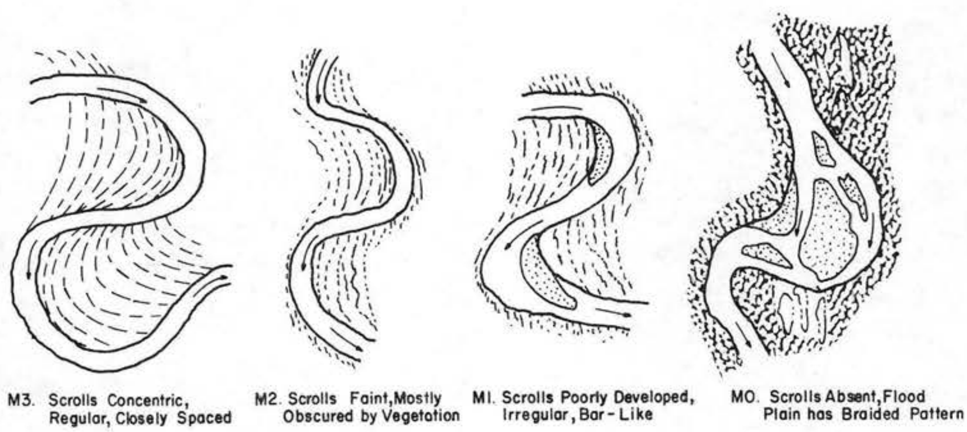


(c) Types of sinuosities

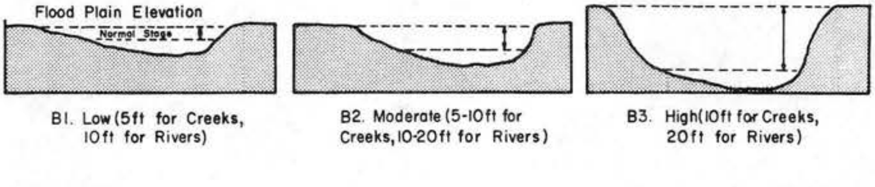
Figure 3-11 Classification of River Channels (after Culbertson et al., 1967).



(d) Oxbow lakes on floodplain

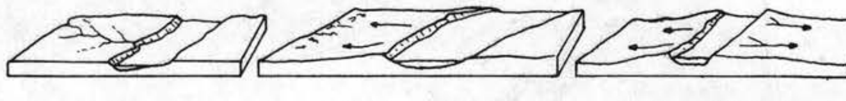


(e) Types of meander scroll formations

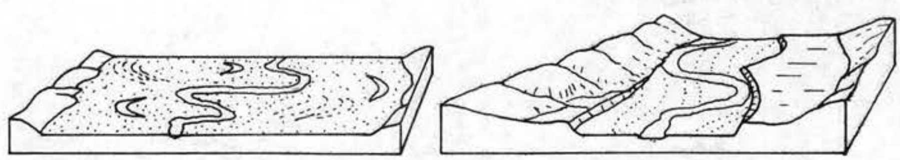


(f) Types of bank heights

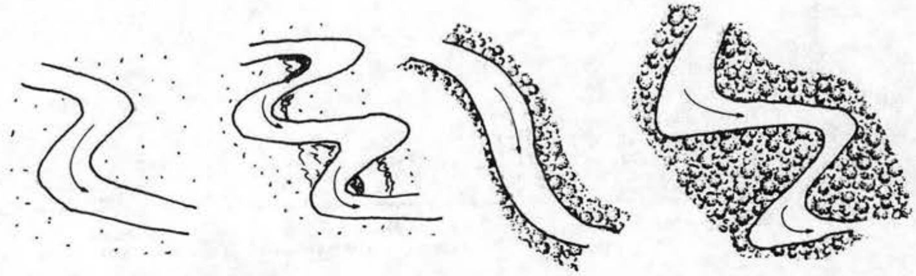
Figure 3-11 Classification of River Channels (after Culbertson et al., 1967) (Continued).



(g) Types of natural levee formations



(h) Types of modern floodplains



(i) Types of vegetal patterns

Figure 3-11 Classification of River Channels (after Culbertson et al., 1967) (Continued).

narrow and confined by terraces or valley walls the river flowing therein is usually mature. Typical vegetative patterns that are observed along meandering channels are shown in Figure 3-11i. In general, the growth of vegetation is indicative of the presence of silts and clays in the river banks and the floodplain. This is particularly true if the floodplain is well drained. With good drainage the silt and clay are essential to the growth of vegetation because of their water holding capability.

3.3 Qualitative Response of River Systems

Many rivers have achieved a state of approximate equilibrium throughout long reaches. For practical engineering purposes, these reaches can be considered stable and are known as "graded" streams by geologists and as "poised" streams by engineers. However, this does not preclude significant changes over a short period of time or over a period of years. Conversely, many streams contain long reaches that are actively aggrading or degrading.

Regardless of the degree of channel stability, man's local activities may produce major changes in river characteristics both locally and throughout an entire reach. All too frequently the net result of a river improvement is a greater departure from equilibrium than that which originally prevailed. Good engineering design must invariably seek to enhance the natural tendency of the stream toward poised conditions. To do so, an understanding of the direction and magnitude of change in channel characteristics caused by the actions of man and nature is required. This understanding can be obtained by: (1) studying the river in a natural condition, (2) having knowledge of the sediment and water discharge, (3) being able to predict the effects and magnitude of man's future activities, and (4) applying to these a knowledge of geology, soils, hydrology, and hydraulics of alluvial rivers.

Predicting the response to channel development is a very complex task. There are a large number of variables involved in the analysis that are interrelated and can respond to changes in a river system and i the continual evolution of river form. The channel geometry, bars, and forms of bed roughness all change with changing water and sediment discharges. Because such a prediction is necessary, useful methods have been developed to predict both qualitative and quantitative response of channel systems to change.

3.3.1 Prediction of General River Response to Change

Quantitative prediction of response can be made if all of the required data are known with sufficient accuracy. Usually, however, the data are not sufficient for quantitative estimates, and only qualitative estimates are possible.

The response of channel pattern and longitudinal gradient to variation in selected parameters has been discussed in previous sections In more general terms, Lane (1955) 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 (y) 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/y) 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 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.

$$Q_s \sim \frac{(\tau_o V) W C_F}{D_{50}} \tag{3.6}$$

where: τ_o = bed shear

V = cross-sectional average velocity

C_F = concentration of fine material load

Equation (3.6) can be modified by substituting $\gamma y S$ for τ_o, and:

$$Q = AV = WyV \tag{3.7}$$

from continuity, yielding

$$Q_s \sim \frac{(\gamma y S) WV}{D_{50}/C_F} = \frac{\gamma QS}{D_{50}/C_F} \tag{3.8}$$

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

$$QS \sim Q_s D_{50} \tag{3.9}$$

Equation (3.9) is essentially the relation proposed by Lane (1955), except fall diameter, which includes the effect of temperature on transport, has been substituted for the physical median diameter used by Lane.

3.3.2 Applications of Qualitative Analysis

Equation (3.9) 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 3-12). Aggradation in the reservoir

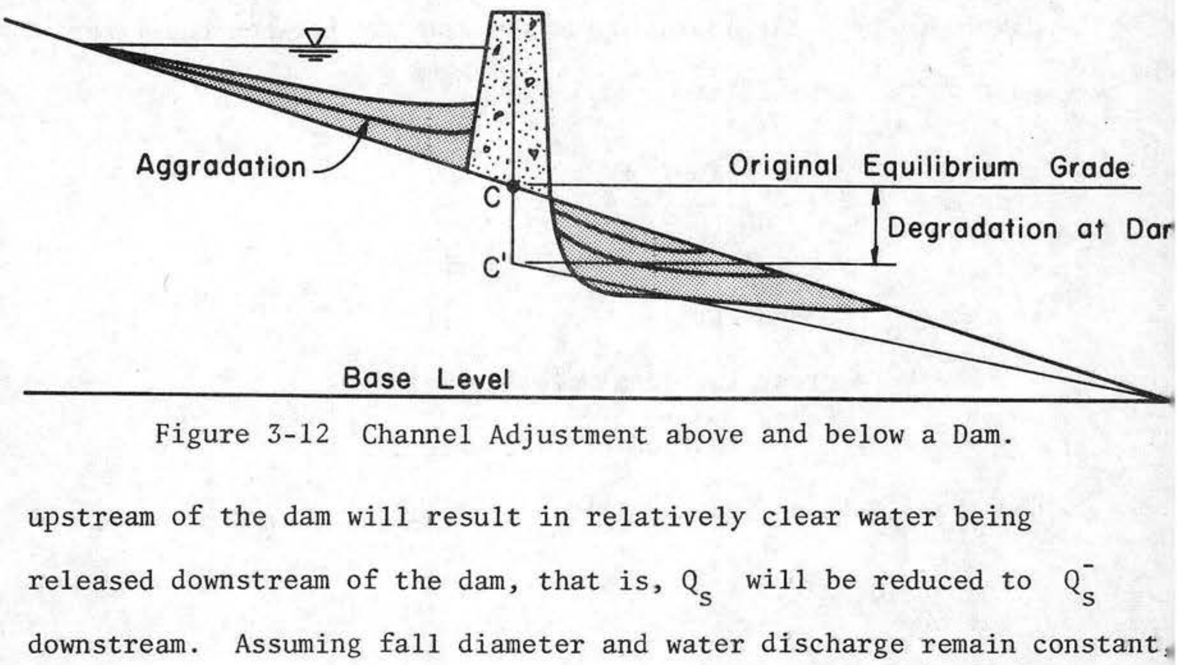


Figure 3-12 Channel Adjustment above and below a Dam.

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 (3.9)

$$Q_s^- D_{50}^o \sim Q^o S^- \qquad (3.10)$$

In Figure 3-12 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.

As another example, consider a tributary entering the main river at point C that is relatively small but carries a large sediment load (Figure 3-13). This increases the sediment discharge in the main stream from Q_s to Q_s^+. It is seen from Equation (3.9) that, for a significant increase in sediment discharge (Q_s^+) the channel gradient (S)

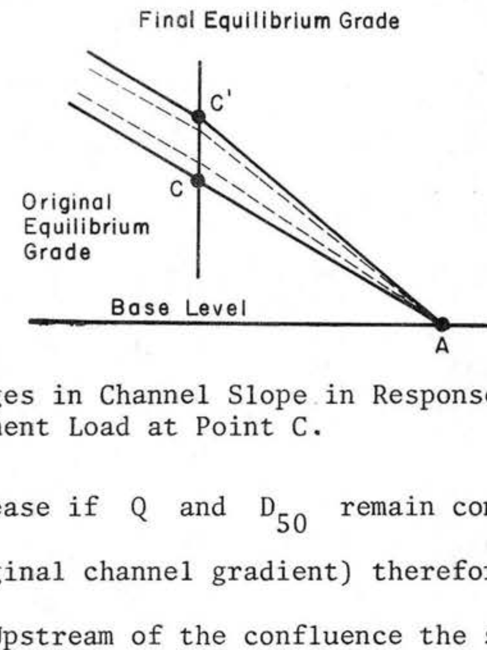


Figure 3-13 Changes in Channel Slope in Response to an Increase in Sediment Load at Point C.

below C must increase if Q and D_{50} remain constant. The line CA (indicating the original channel gradient) therefore changes with time to position C'A. Upstream of the confluence the slope will adjust over a long period of time to the original channel slope. The river bed will aggrade from C to C'. This same result was obtained in Section 3.2.7 concerning the dynamic nature of a stream's longitudinal profile (Figure 3-10).

The engineer is also interested in quantitative results in addition to qualitative indications of trends. The geomorphic relation $QS \sim Q_s D_{50}$ is only an initial step in analyzing long-term channel response problems. However, this initial step is useful, because it warns of possible future difficulties in designing channel improvement and flood protection works and provides a good first-order estimate of response to development.

3.3.3 Hydraulic Geometry of Alluvial Channels

Hydraulic geometry is a general term applied to alluvial channels to denote relationships between discharge, Q, and the channel morphology, hydraulics and sediment transport. In self-formed alluvial channels, the morphologic, hydraulic and sedimentation characteristics of the channel are determined by a large variety of factors. The mechanics of such factors is not fully understood. However, alluvial streams do exhibit some quantitative hydraulic geometry relations. In general, these relations apply to channels within a physiographic region and can be derived from data available on gaged rivers. It is understood that hydraulic geometry relations express the integral effect of all the hydrologic, meteorologic, and geologic variables in a drainage basin.

The hydraulic geometry relations of alluvial streams are useful in river engineering. The forerunner of these relations are the regime theory equations of stable alluvial canals. A generalized version of hydraulic geometry relations was developed by Leopold and Maddock (1953) for different regions in the United States and for different types of rivers. In general the hydraulic geometry relations are stated as:

$$W = a\,Q^b$$

$$y_o = c\,Q^f$$

$$V = k\,Q^m$$

$$Q_T = p\,Q^j$$

$$S = t\,Q^z$$

$$n = r\,Q^y$$

where W is the channel width, y_o is the channel depth, V is the average velocity of flow, Q_T is the total bed-material load, S is the energy gradient, n is the Manning's roughness coefficient, and Q is the discharge as defined in the following paragraphs. The coefficients a, c, k, p, t, r and exponents b, f, m, j, z, y in these equations are determined from analysis of available data on one or more streams. From the definition equation $Q = Wy_oV$ (continuity), it is seen that

$$a \cdot c \cdot k = 1$$

and

$$b + f + m = 1$$

Leopold and Maddock (1953) have shown that in a drainage basin, two types of hydraulic geometry relations can be defined: (1) relating W, y_o, V and Q_T to the variation of discharge at a station, and (2) relating these variables to the discharges of a given frequency of occurrence at various stations in a drainage basin. Because Q_T is not available they used Q_s the suspended-load transport rate. The former are called <u>at-station</u> relationships and the latter <u>downstream</u> relationships. The distinction between at-station and downstream hydraulic geometry relations is illustrated in Figures 3-14 and 3-15.

More recently hydraulic geometry relations were theoretically developed at CSU. These relations are almost identical to those

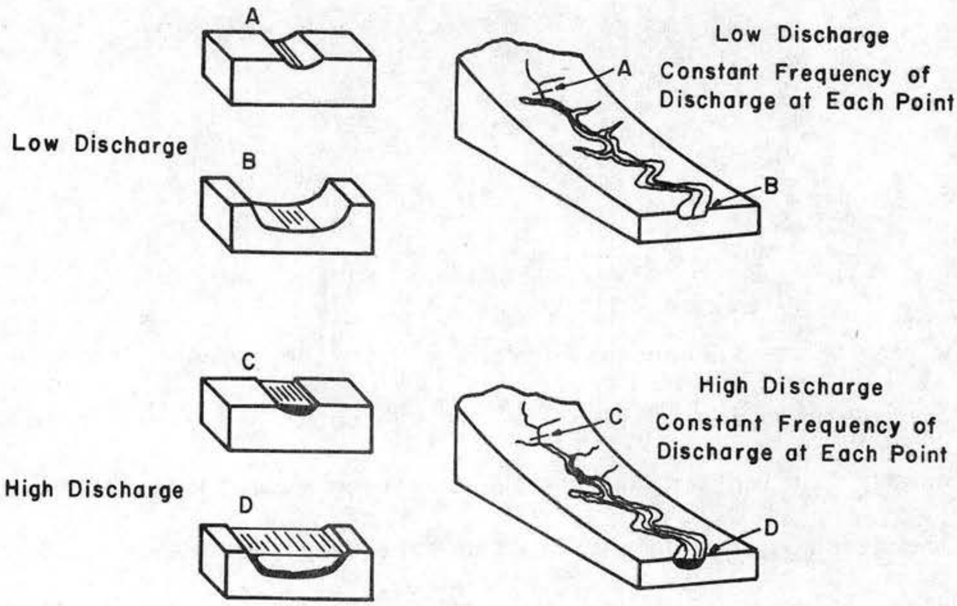


Figure 3-14 Variation of Discharge at a Given River Cross Section and at Points Downstream (after Leopold and Maddock, 1953). At-Station Relations Pertain to Individual Sites such as A or B. Downstream Relations Pertain to a Channel (Segmen A-B) or Drainage Network for Discharge of a Given Frequenc of Occurrence.

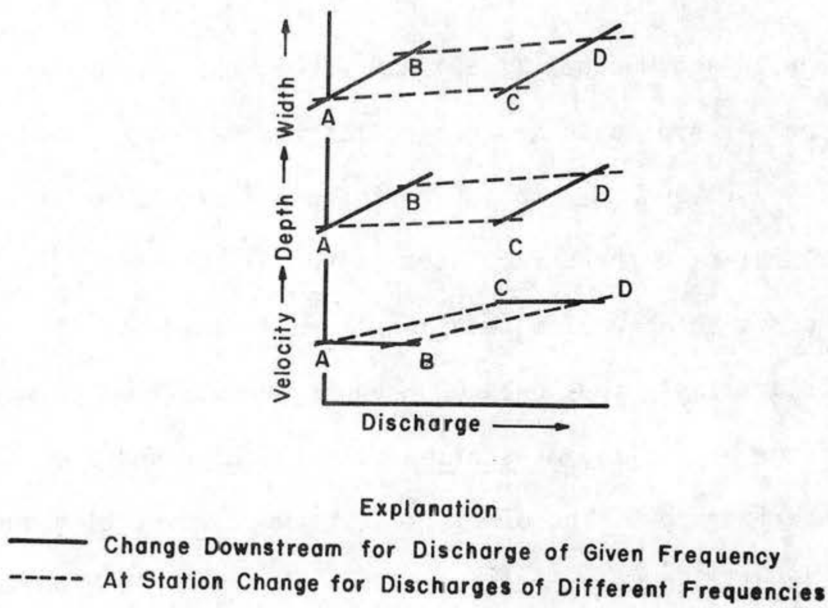


Figure 3-15 Schematic Variation of Width, Depth, and Velocity with At-Station and Downstream Discharge Variation (after Leopold and Maddock, 1953).

proposed by Leopold and Maddock. The at-station relations derived at CSU are:

$$W \sim Q^{0.26} \tag{3.11}$$

$$y_o \sim Q^{0.46} \tag{3.12}$$

$$S \sim Q^{0.00} \tag{3.13}$$

$$V \sim Q^{0.30} \tag{3.14}$$

Equation (3.13) implies that slope is constant at a cross section. This is not quite true. At low flow the effective channel slope is that of the thalweg that flows from pool through crossing to pool. At higher stages the thalweg straightens somewhat shortening the path of travel and increasing the local slope. In the extreme case river slope approaches the valley slope at flood stage. It is during high floods that the flow often cuts across the point bars developing chute channels. This path of travel verifies the shorter path the water takes and that a steeper channel prevails under this condition.

The derived downstream relations for bankfull discharge are:

$$y_b = Q_b^{0.46} \tag{3.15}$$

$$W_b = Q_b^{0.46} \tag{3.16}$$

$$S = Q_b^{-0.46} \tag{3.17}$$

$$V_b = Q_b^{0.08} \tag{3.18}$$

Here the subscript b indicates the bankfull condition.

3.3.4 Prediction of Channel Response to Change

In Section 3.3.2 it was illustrated that Equation (3.9) could be used to predict changes in channel profiles caused by changes in water and sediment discharge. It is now possible to talk qualitatively about changes in channel profile, changes in river form, and changes in river cross section both at a section and along the river channel using the other relations presented above.

This can be best illustrated by application. Referring to Table 3- consider the effect of an increase in discharge indicated by a plus sign on line (a) opposite discharge. The increase in discharge may affect the river form, energy slope, stability of the channel, cross-sectional area and river stage. Equations (3.2) and (3.3) or Figure 3-7 show that an increase in discharge could induce a change in the direction of a braided form. Whether or not the channel form changed would depend on the river character prior to the increase in discharge. With the increase in discharge the stability of the channel would be reduced according to Equation (3.14) which indicates an increase in velocity. On the other hand, this prediction could be affected by changes in form of bed roughness that dictate resistance to flow.

From Chapter 2 recall that the wash load increases the apparent viscosity of the water and sediment mixture. This makes the bed material behave as if it were smaller. In fact, the fall diameter of the bed-material is made smaller by significant concentrations of wash load. With more wash load, the bed material is more susceptible to transport and any river carrying significant wash load will change from lower to upper regime at a smaller Froude number than otherwise. Also, the viscosity is affected by changes in temperature.

Seepage forces resulting from seepage outflow help stabilize the channel bed and banks. With seepage inflow, the reverse is true. Vegetation adds to bank stability and increases resistance to flow reducing the velocity. Wind can retard flow increasing roughness and depth when blowing upstream. The reverse is true with the wind blowing

Table 3-1 Qualitative Response of Alluvial Channels

Variable		Change in Magnitude of Variable	Effect on: Regime of Flow	River Form	Resistance to Flow	Energy Slope	Stability of Channel	Area	Stage
Discharge	(a)	+	+	M→B	±	-	-	+	+
	(b)	-	-	B→M	∓	+	+	-	-
Bed-Material Size	(a)	+	-	M→B	+	+	±	+	+
	(b)	-	+	B→M	-	-	±	-	-
Bed-Material Load	(a)	+	+	B→M	-	-	+	-	-
	(b)	-	-	M→B	+	+	-	+	+
Wash Load	(a)	+	+		-	-	±	-	-
	(b)	-	-		+	+	±	+	+
Viscosity	(a)	+	+		-	-	±	-	-
	(b)	-	-		+	+	±	+	+
Seepage force	(a)	Outflow	-	B→M	+	-	+	+	+
	(b)	Inflow	+	M→B	-	+	-	-	-
Vegetation	(a)	+	-	B→M	+	-	+	+	+
	(b)	-	+	M→B	-	+	-	-	-
Wind	(a)	Downstream	+	M→B	-	+	-	-	-
	(b)	Upstream	-	B→M	+	-	-	+	+

downstream. The most significant result of wind effect is wind generated waves and their adverse effect on channel stability.

In many instances it is important to assess the effects of changes in water and sediment discharge on specific variables such as depth of flow, channel width, characteristics of bed materials, velocity and so forth. For this type of analysis we can use Equation (3.9) and the at-station hydraulic geometry relation. Equation (3.9) can be written in terms of width, depth, velocity, concentration of bed-material discharge C_s, and water discharge Q or

$$QS \propto Q_s D_{50} = QC_sD_{50} \tag{3.19}$$

and

$$C_s D_{50} \propto S \tag{3.20}$$

These equations are helpful for detailed analysis.

3.4 The Crossing and Pool Sequence

The crossing and pool sequence is common to both meandering reaches (Figure 3-1c) and straight reaches with a thalweg that meanders through alternate bars (Figure 3-1b). 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 3-1). 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 3-16c) with the deepest points located near the outer (concave) bank and shallow point bars located on the inner (convex) bank (Figure 3-16a). In the

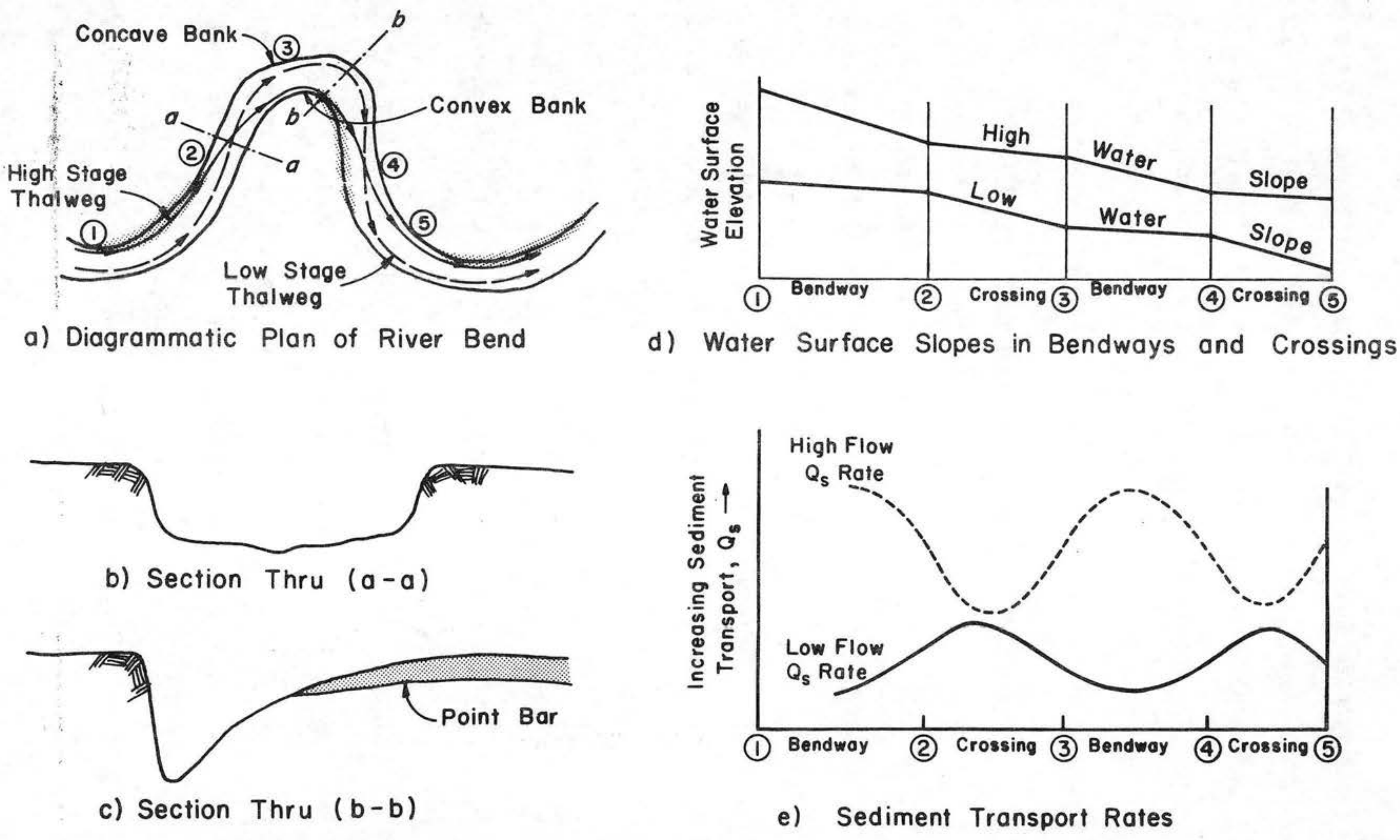


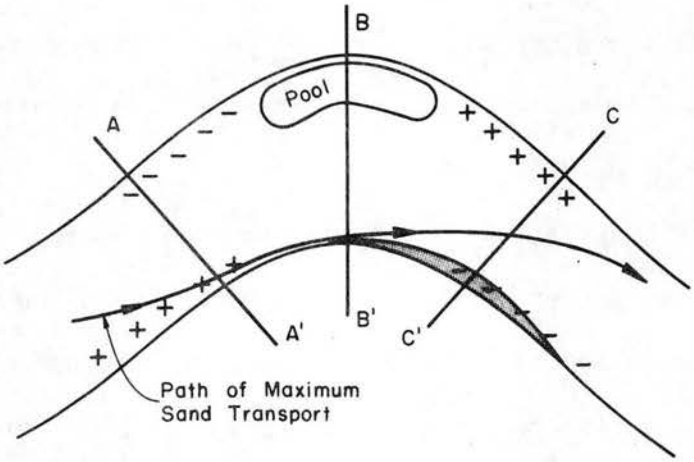
Figure 3-16 Characteristics of a River Bendway.

transition zone between bends, flow lines straighten and the cross section takes the form of a wide, shallow trough (Figure 3-16b) 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.

3.4.1 Morphology of the Crossing and Pool Sequence

As noted in Section 2.1, the existence of transverse secondary currents provides a good first order explanation of the morphology of a bendway, however, 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, it is sediment distribution, not secondary currents, which is primarily responsible for the development of a point bar.

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. The zone of maximum shear stress, and, thus, maximum sediment discharge, 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 3-17a), 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



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 3-17 Influence of Secondary Currents, Shear Stress, and Sediment Distribution on Bendway Morphology.

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 3-17 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 3-17a. The bend is divided 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 3-17b.

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 investigations 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).

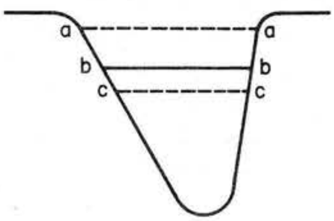
As a result of the asymmetric distribution of zones of deposition and erosion, a meander loop will tend to migrate laterally across the floodplain and in a downstream direction. The outer bank of the bend recedes through bankline erosion, and the inner bank follows as the point bar deposit grows (Figures 3-4 and 3-5), 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.

3.4.2 The Influence of Stage

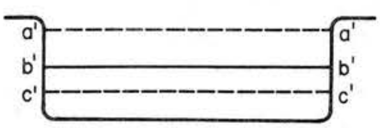
The influence of river stage on the morphology of the crossing and pool sequence has been recognized by many investigators. In regard to maintaining navigable depths by dredging, Ockerson (1898) observed: "It is not easy to find a satisfactory explanation as to why sediment piles up in ridges instead 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 3-16. Comparison of the change in flow area at different stages for cross sections in pools and crossings (Figure 3-18) with the change in water surface



a) At a Bend



b) At a Crossing

Figure 3-18 Variation in Stage at Typical Cross Sections (after Lane and Borland, 1954).

profiles (Figure 3-16d) provides an explanation of the phenomena observed by Ockerson. At intermediate stages (lines b-b and b'-b' Figure 3-18) 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.

A change in stage is accompanied by a reversal of the processes of erosion and deposition on the crossings and in the pools of a meandering thalweg bendway. In addition, the scale of these processes is significantly different at high and low stage. As sketched in Figure 3-16e, at high flow the lower sediment transport rate on the crossing indicates a tendency to deposit, while the higher transport rate in the pool indicates a tendency to erode. Because of the higher flow velocities and greater volumes of water that accompany high-stage flow, the quantities of sediment that are both scoured and deposited greatly exceed quantities of sediment in motion at low-stage flow (Figure 3-16e) As a result low flow scour on a crossing is generally not sufficient to remove the material deposited during high-stage flow. In the meandering thalweg system, then, the crossings act as sediment "source" areas and the pools act as sediment "sink" areas.

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 3-16a). 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.

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 development programs such as navigation channel maintenance. 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 either its character or capacity.

3.5 Divided Flow Reaches

In his classic paper on the geology of the Hardin and Brussels quadrangles (in Illinois) Rubey (1952) notes that the importance of islands in the morphology of large rivers like the Mississippi has been neglected by most writers. A survey of European literature also leads to the conclusion that "little has been published about the origin, shape and life of islands in alluvial rivers in their natural state" (NEDECO, 1959). This section summarizes the basic concepts and recent research relative to the island and side channel configuration of divided flow reaches on the Mississippi River.

3.5.1 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.

3.5.2 Evolution of Islands and Side Channels

As is established in Chapter 4, 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 large tributaries, the Cuivre River and Peruque and Dardenne Creeks, respectively. 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 5-12).

The coincidence 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

*River Miles are measured from the mouth of the Ohio River and will be abbreviated RM throughout this handbook.

one will build high enough to escape destruction for several seasons. Once vegetation appears, island growth progresses 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 evolutior 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. 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 (Simons et al., 1974).

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 alon the Missouri bank. Shull revisited the region in 1933 and reported tha 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 numerous timber snags common to the Mississippi. The deltaic sandbars at the mouths of tributaries, which Rubey cites as the 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. 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 River Mile 280 (Figure 5-16). The 1880 and 1927 outlines in Figure 5-16 were made at stages so low that the island interior was not inundated, but the 1973 soundings were made at a higher stage, revealing the crab claw signature that reflects the processes of island building in the river (Lagasse, 1975).

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):

> "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 3 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.

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 3-19) is a naturally contracted reach where the width is controlled by rock outcroppings. In Figure 3-19 Thebes Gap is the reach between Grid Miles 0 and 3. In 1884 there were six islands in the

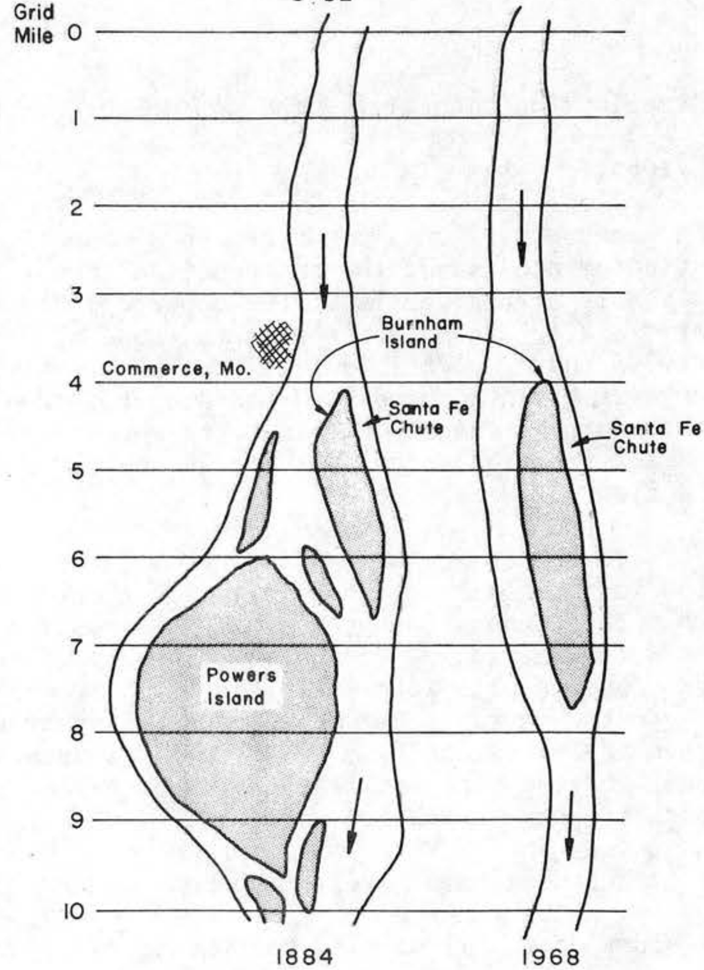


Figure 3-19 Loss of Chute Channels below Thebes Gap.

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 of Santa Fe Chute 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 3-16) 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. 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 can also produce the main channel, island, and chute channel configuration.

Simons, Schumm, and Stevens (1974) 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 <u>inside</u> of the bend and Picayune Chute and a smaller chute channel on the outside of the bend (Figure 3-20a). The configuration of the bend suggests that a point bar cutoff had occurred in the past.

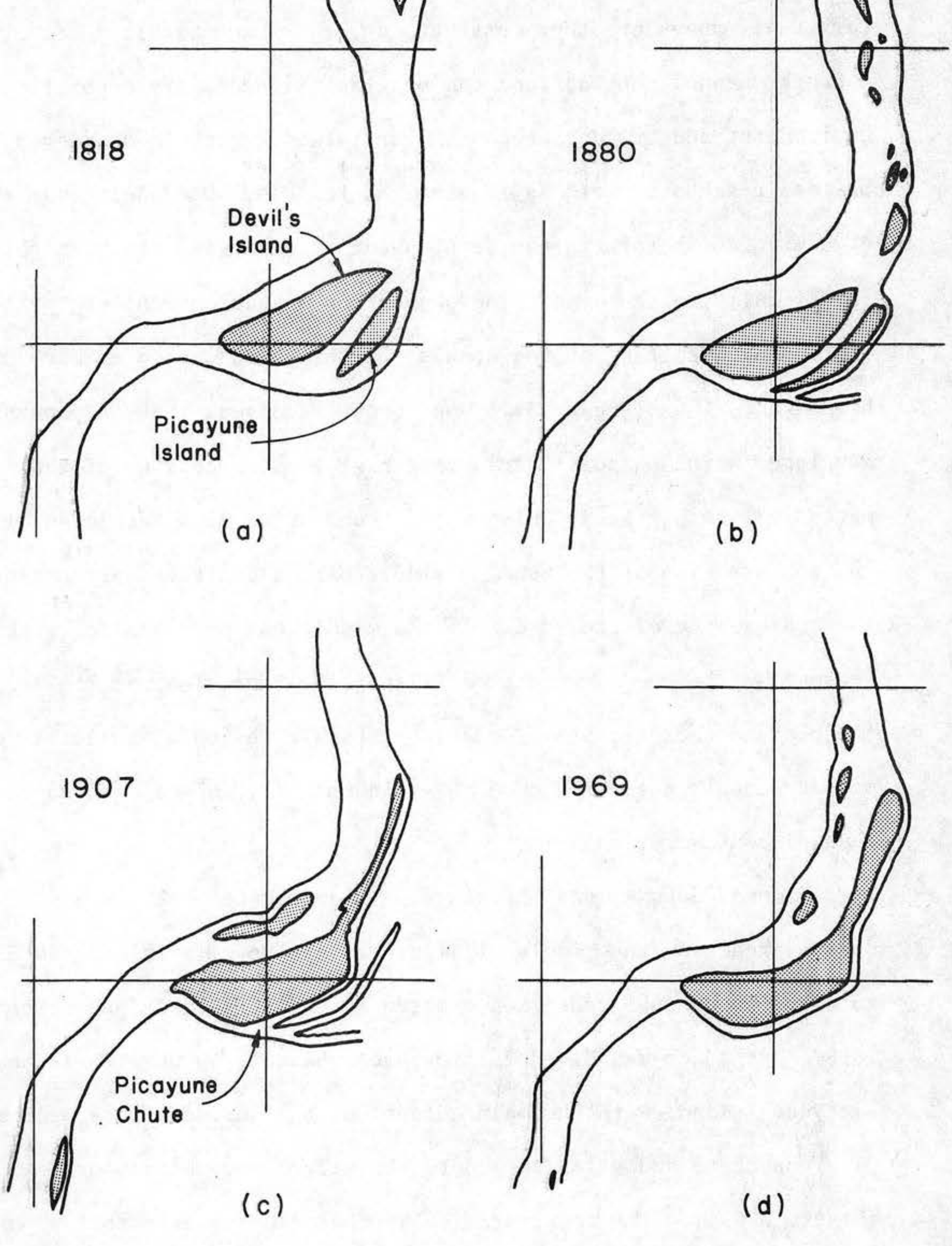


Figure 3-20 Devil's Island and Picayune Chute, 1818-1969.

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 3-20b). 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 and the second chute channel closed at the head and became a slough (Figure 3-20c). 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 3-20d).

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.

After a detailed study of the evolution of natural side channels on the Middle Mississippi Simons, Schumm, and Stevens concluded that in its natural state, an alluvial river divides itself into two or more channels by the processes of either erosion or deposition. The side channels so formed can grow in size and capture most of the discharge and become the main channel; they can deteriorate in size and become a part of the floodplain; or they can grow to the size of the main channel and maintain that size. In the natural state, those side channels which are obliterated by deposition are replaced by new side channels caused by floods and/or river migrations.

In the Middle Mississippi, the river is no longer free to migrate and produce new side channels. There are no meander loops to be cutoff by floods. Except for the major chute channels, natural side channels in the Middle Mississippi River are being filled with sediment. Most of the major chute channels have achieved a size which indicates they could exist for a long period of time.

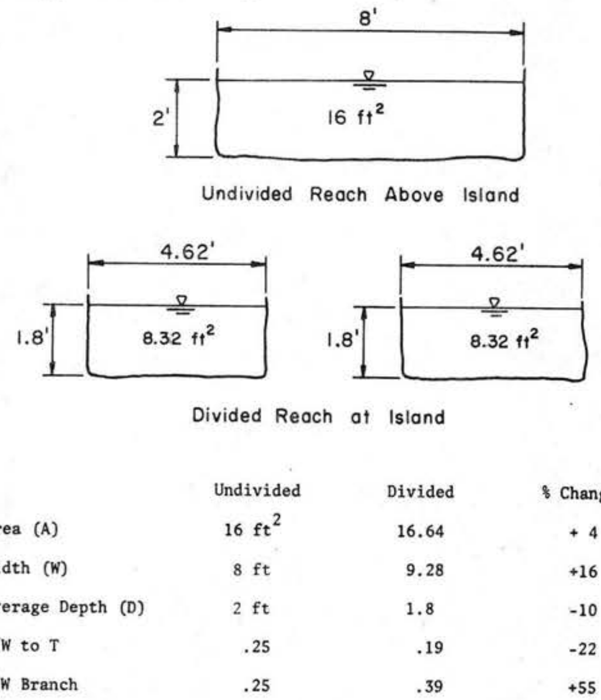
In the absence of further man-induced changes in the hydrology or geomorphology of the Middle Mississippi River, all natural side channels except major chutes, may disappear from the river scene. Within 100 to 200 years, even Picayune Chute (Figure 3-20) will fill. There will be no natural replacement side channels. The preservation of existing side channels should be considered in planning future contraction works in order to maximize environmental benefits and minimize flood stages.

3.5.3 The 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 response of an alluvial river to development. 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. Results of Rubey's investigations are applied to simplified cross sections in Figure 3-21



	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 3-21 Change in Channel Characteristics from Undivided to Divided Reach Based on Rubey's (1952) Data.

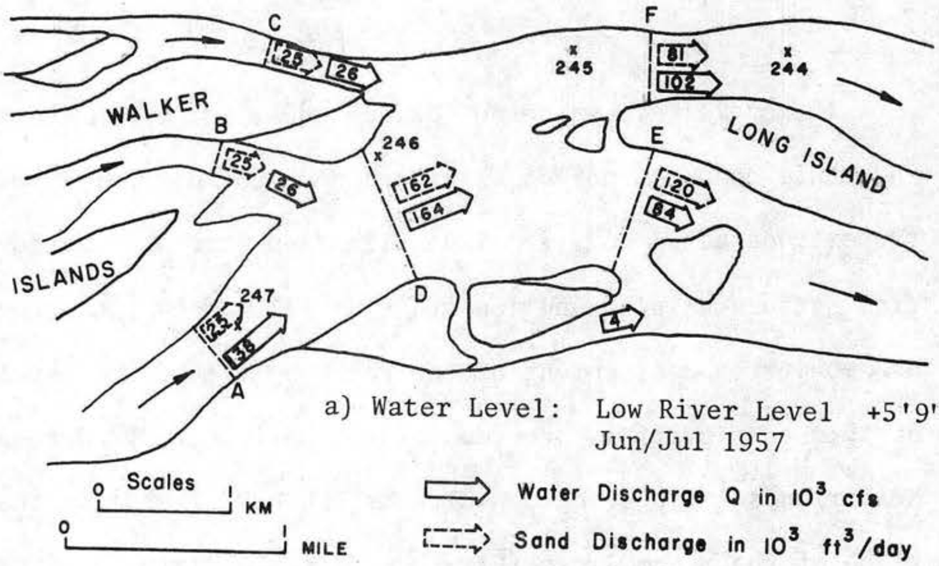
using the average percent change of hydraulic factors observed by Rubey. A number of significant changes are apparent. The total water surface width of the two channels of a divided reach was 16 percent greater than the water surface width of the undivided reach immediately upstream. The average depth 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 an aggrading stream, then, division into several channels would provide a means of optimizing transport capacity and maintaining a graded condition.

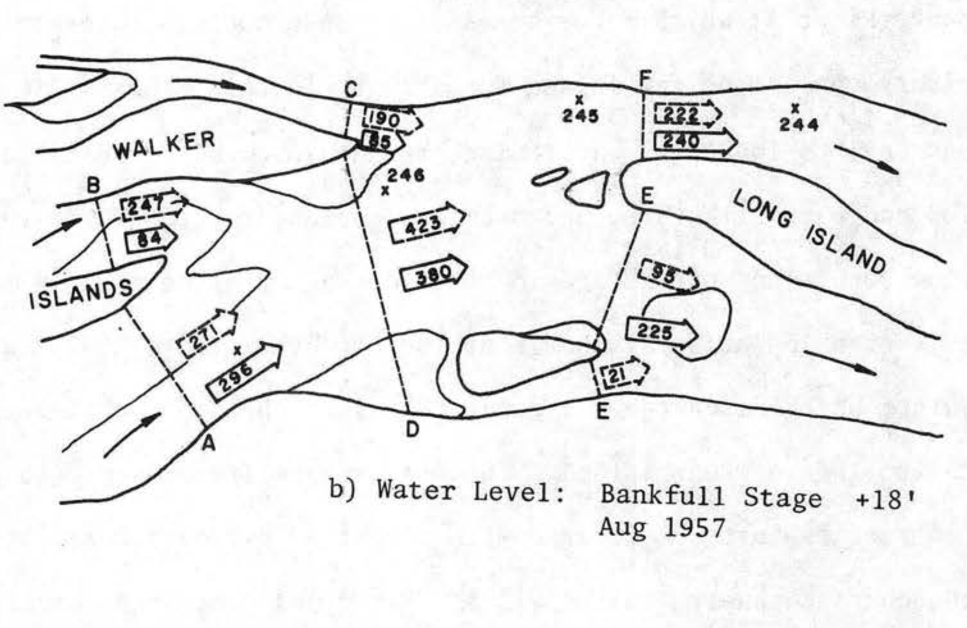
The interdependence of the hydraulic and geomorphic variables of alluvial channel flow is nowhere more apparent than in a divided reach. The increase in wetted perimeter (Figure 3-21) provides a larger frictional surface and, thus, greater resistance to flow in the divided reach. Rubey also indicated a decrease in velocity and an increase in slope through the divided reach. 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 the Netherlands Engineering Consultants (NEDECO) (1959) on the Long Island reach of the Niger River (Figure 3-22). 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 3-22a) 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



a) Water Level: Low River Level +5'9"
Jun/Jul 1957



b) Water Level: Bankfull Stage +18'
Aug 1957

Figure 3-22 Sand Transport and Water Discharge near Long Island, Niger River, Nigeria (after NEDECO, 1959).

places the heavy sediment concentrations along the inside of the bend (Section 3.4.1).

Bankfull stage alters conditions in the divided reach significantly (Figure 3-22b). 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.

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 deposit at low and high stage upstream of the divided reach would create serious problems relative to the maintenance of a navigation channel through the reach.

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 3-23 the average slope of the chute

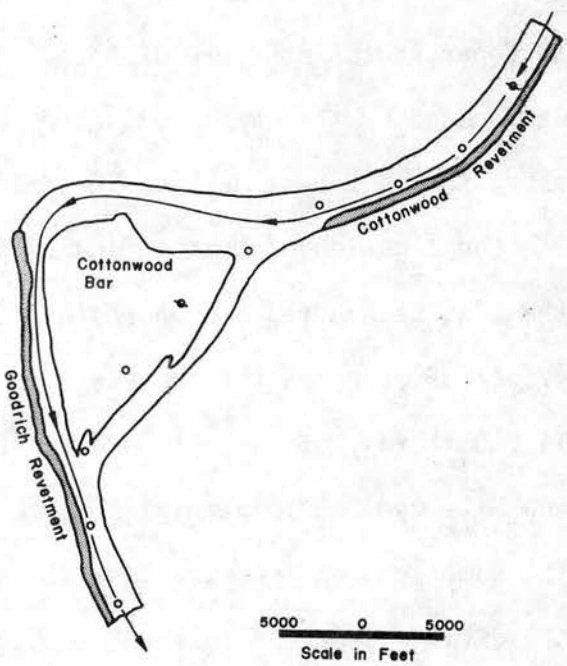


Figure 3-23 Divided Reach--Mississippi River (after Haas, 1963).

channel across the point bar may be twice that of the bendway 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.

Chapter 4

THE GEOMORPHIC PERSPECTIVE

4.1 Introduction

The dynamic nature of river systems has been sketched in Section 1.3. Rivers in the Upper Mississippi River Basin, including the mainstem river itself, are no exception to the rule that in alluvial river systems banks will erode, sediments will be deposited, and floodplains, islands, and side channels will undergo modification with time. Where previous chapters have examined the basic principles of river morphology and river mechanics in a very general manner, this chapter views the geomorphology of a specific system, the Mississippi River above the mouth of the Missouri, in the perspective of geologic time. While interesting in itself, a survey of the geologic scene of the Upper Mississippi River basin supports a discussion of the morphology of the Upper Mississippi prior to man's intervention. In turn, an understanding of the characteristics of the natural river is essential to the analysis of the response of the Upper Mississippi to such development activities as channel stabilization, construction of locks and dams, and dredging. It is an understanding of this response which is the primary objective of the succeeding three chapters.

4.2 The Geologic Scene

4.2.1 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 Ozard Plateau and enters the Mississippi Embayment of the Coastal Plains province just above Cairo (Figure 4-1). In general, the basin is underlain by the

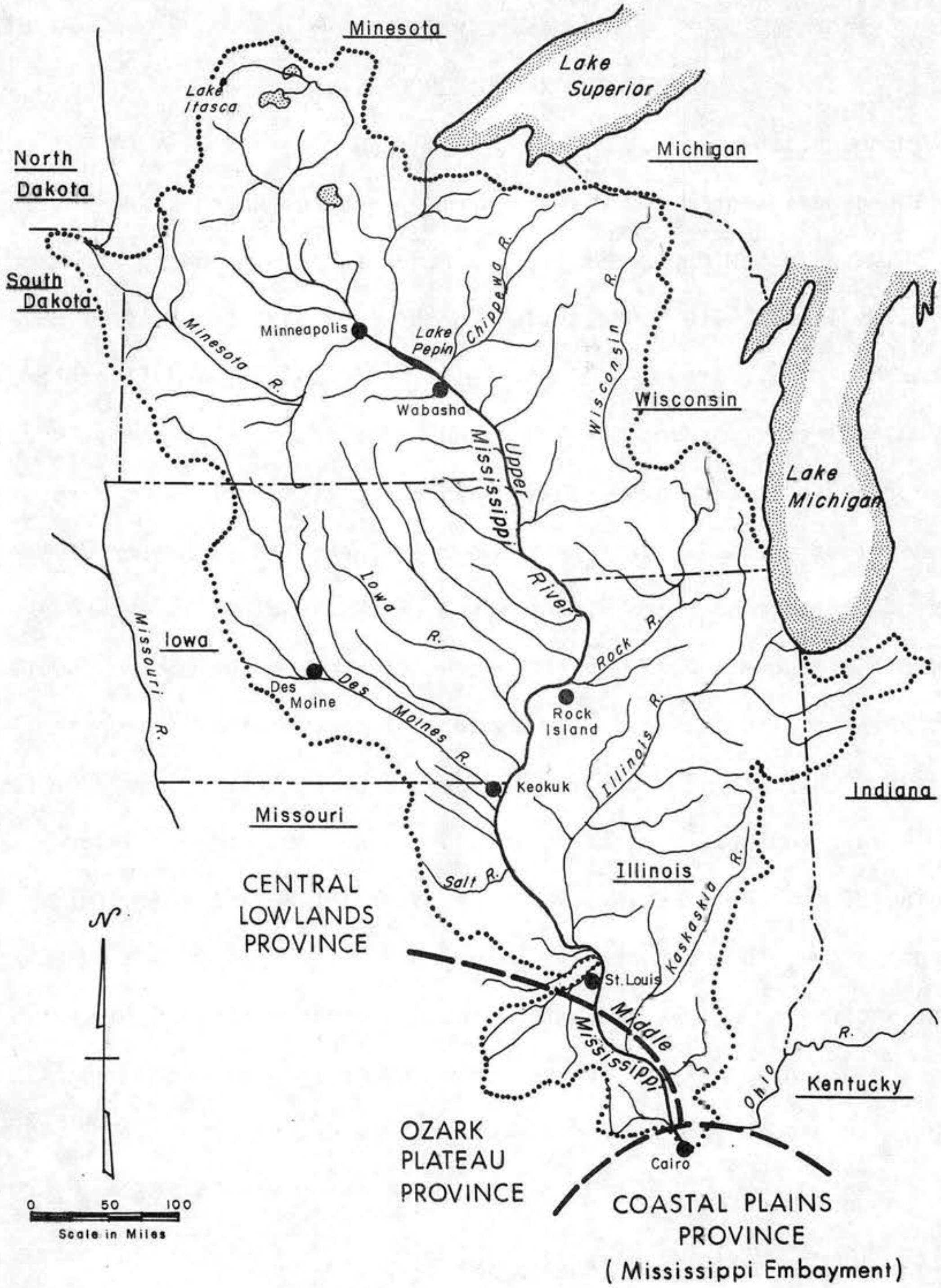


Figure 4-1 Upper and Middle Mississippi River Basin--Physiographic Provinces and Tributaries.

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.

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 (Rubey, 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 4-2). 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).

4.2.2 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

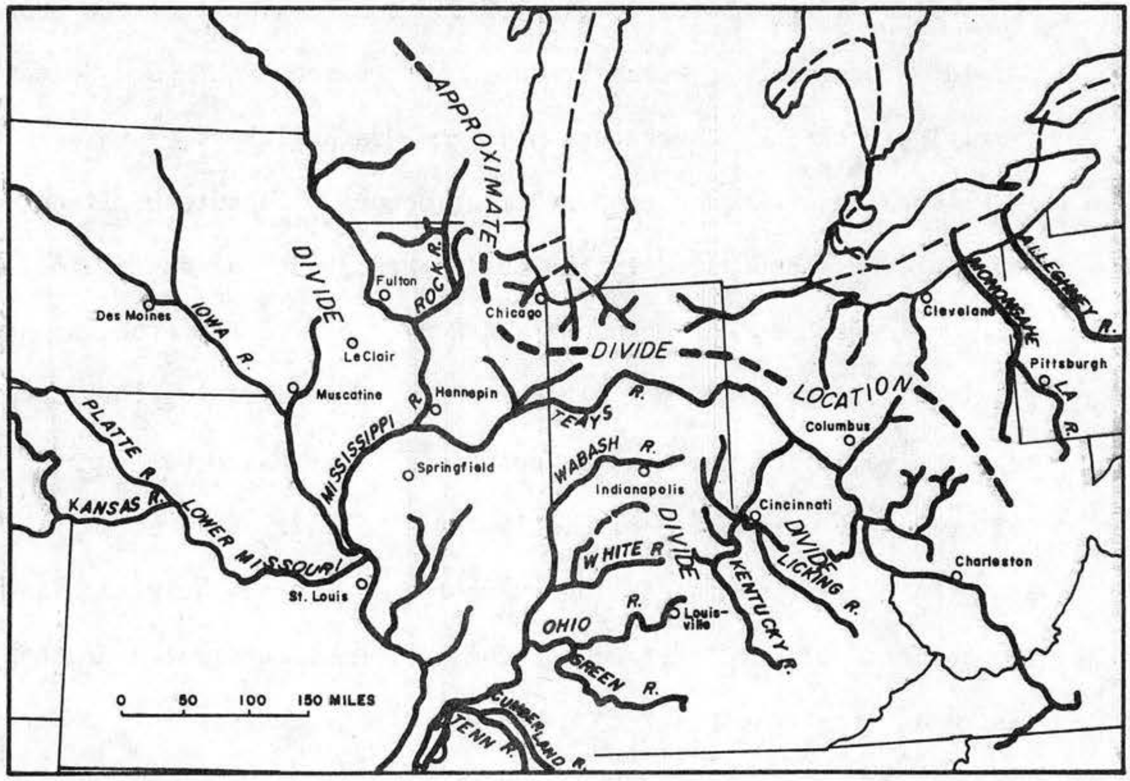


Figure 4-2 Preglacial Map of the River Systems (after Thornbury, 1965).

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.

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 4-3a. The Kansan continental glaciation (Figure 4-3b) 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

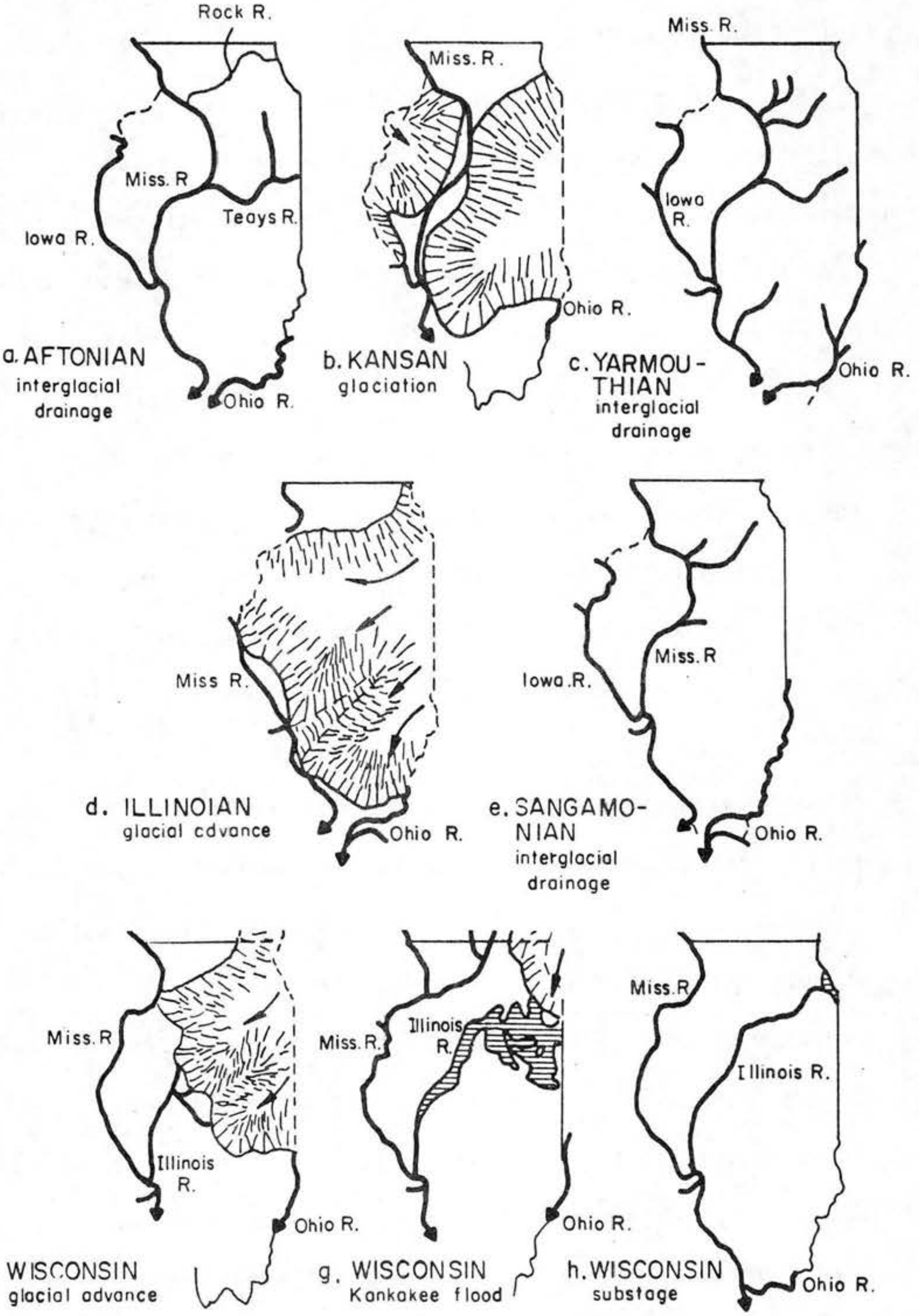


Figure 4-3 Pleistocene Changes--Mississippi River (after Frye et al., 1965).

Illinois valley. This large glacial river cut a deep bedrock 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 reestablished, with the ancient Mississippi River occupying the Illinois valley and the ancestral Iowa River reoccupying the present Mississippi River valley (Figure 4-3c). Then, during the Illinoian glaciation (Figure 4-3d), 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 4-3e) the Mississippi reoccupied the Illinois valley and the Iowa drained through the present Mississippi valley. The final advance of the Wisconsin ice (Figure 4-3f) 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 4-3g) 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 reestablished. 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.

Drilling logs from borings within the floodplain confirm the existence of a deeply buried bedrock valley underlying these alluvial deposits. Drilling logs also verify the existence of progressively coarser granular material with depth, and this alluvial sequence, with its upward decrease in particle size provides evidence 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 4-1) terraces up to 100 feet high are located adjacent to the present river floodplain, providing a nearly inexhaustible source of coarse sediments. 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. The close of the Wisconsin glaciation saw the establishment of the major features of today's drainage pattern in most of the Upper Mississippi basin (Figure 4-3h).

4.2.3 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 4-1). 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, in a 1952 paper on "The Lakes of Minnesota--Their Origin and Classification" compiled a summary of late-glacial and post-glacial events which led to the development of Lake Pepin. The development sequence is sketched in Figure 4-4 and outlined below:

a. Loss of volume in the Mississippi River because of the change of outlet of Glacial Lake Agassiz, permits the growth of a delta at the mouth of the Chippewa River.

b. The continued growth of the Chippewa delta causes early Lake Pepin to extend upstream to St. Paul. A delta forms at the head of this lake, and silt and clay deposit in the deeper, quiet waters of early Lake Pepin.

c. The Chippewa delta grows larger and higher, while the delta at the head of the lake advances downstream to the mouth of the St. Croix River, burying the previously deposited silts and clays and blocking the lower end of the St. Croix Valley, thus forming Lake St. Croix.

d. The continued growth of the Chippewa delta raises the level of Lake Pepin, while the advance of the delta below St. Paul continues to cover previously deposited lake silts and clays.

e. Present configuration of sediments in the valley as shown by borings.

4.2.4 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

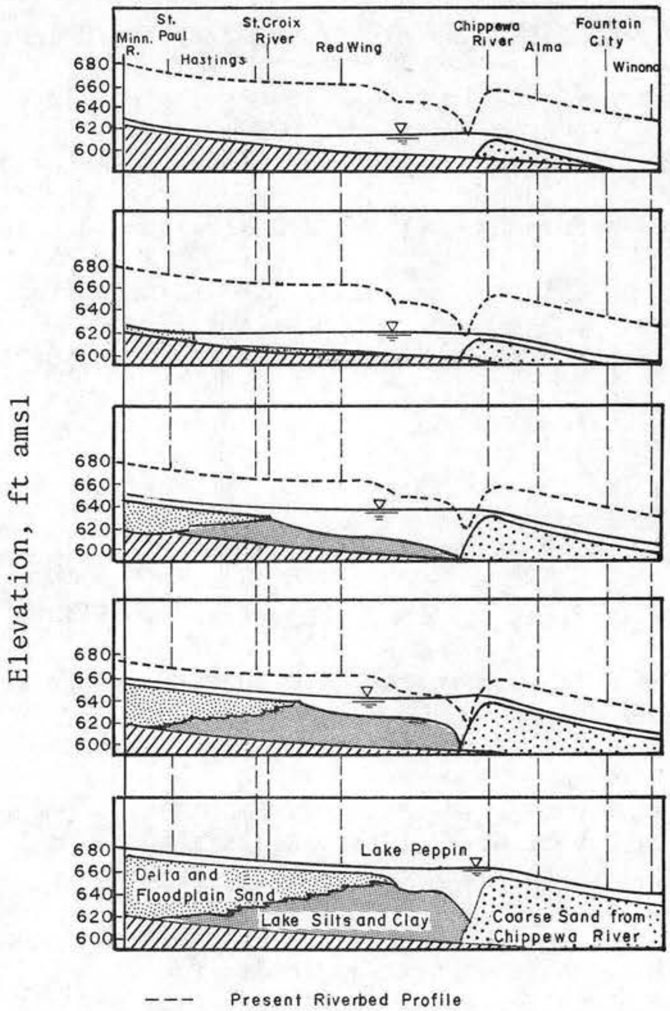


Figure 4-4 Post-Glacial Geologic History of Lake Pepin (after Zumberge, 1952).

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 anticlinal arches, and broad, saucer-like basins modify the prevailing structure and dip of the bedrock. There are three principal examples of these features: the Illinois basin, a large spoon-like 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.

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. 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. 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. 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 4-1) where the river has incised through glacial drift and deeply into the bedrock to form gorges with vertical exposures of bedrock. There is also regional exposure of bedrock in the vicinity of Rock Island, Illinois and Keokuk, Iowa (Figure 4-1).

4.3 The Natural River

4.3.1 Definition

Man's development 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, between St. Louis, Missouri and Cairo, Illinois, 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.

4.3.2 The River

In establishing conditions of the natural river, journals of early explorers provide a detailed, if nontechnical, description. For example, on reaching the junction of the Wisconsin and the Mississippi Rivers (Figure 4-1) 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 4-1), 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 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..."

In regard to the character of the bed of the Mississippi and the 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 mainland by the filling up of their chutes. The process of formation and destruction is interesting. Driftwood 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." Ockerson was also impressed by the dynamic character of the river, particularly in regard to the formation of sandbars which "are numerous, and crossings are consequently frequent, and their locations are constantly shifting."

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 variety 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).

4.3.3 The Morphology of the Natural River

Although a continuum of channel patterns does exist on the Upper Mississippi, descriptions of the natural river support the conclusion that many reaches are braided or "island braided" in character. The conditions which produce braiding on the Upper Mississippi are rather unusual. An understanding of these conditions is essential to an understanding of the character of the modern Mississippi. The influence of the glacial history of the Upper Mississippi River basin on the position and configuration of the valley of the Upper Mississippi has been reviewed (Figure 4-3). As previously noted, the close of the Wisconsin glaciation saw the establishment of the major features of today's drainage pattern in most of the Upper Mississippi basin. Lane (1957) in a study of the conditions that produced braiding on the Upper Mississippi River concludes that this unique glacial history is primarily responsible for the braided character in many portions of the modern river.

Subsequent to the Wisconsin glaciation, the glaciers retreated northward. Eventually, this retreat uncovered the channels of the St. Lawrence and the rivers leading to Hudson Bay, and the water from the lakes such as Lake Agassiz escaped to the sea through these channels, ceasing to flow down the Mississippi. The channel which was excavated by the great flow of water from these glacial lakes, plus the normal runoff of the river watershed however, was on too flat a slope to transport all the sediment brought in from the Mississippi watershed with the smaller flow resulting from the normal runoff from the Upper Mississippi watershed without the glacial water. The great filling of the stream valley then began, and is still under way. In this filling-braided channel with a multiplicity of interlacing channels, islands have been formed as shown in Figure 4-5 which was taken from the Waukon, Iowa and Wisconsin Quadrangle of the U.S. Geological Survey. According to Lane, the slope of the Mississippi River is still too flat to move all of the sediment brought to it, and filling will continue until the slope is steep enough; probably about that which the river had before the glacial period.

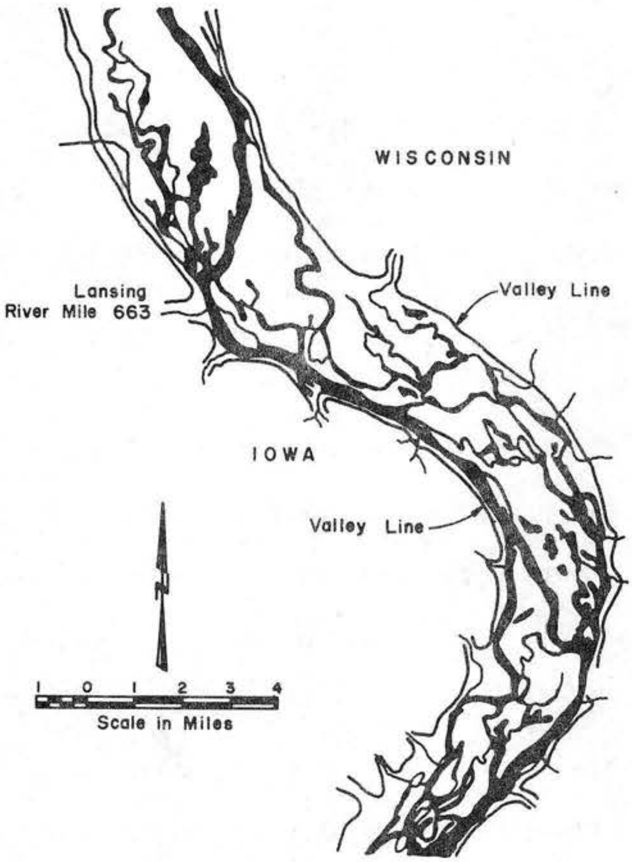


Figure 4-5 A Low-Slope Braided Section of the Upper Mississippi River near Lansing, Iowa (after Lane, 1957).

Lane concluded that the aggrading of this valley must go on very slowly, as it has to fill the entire width of the valley and the annual sediment discharge of the tributaries is small. Most of the sediment deposition probably takes place in the ponds, lakes, and secondary channels, where the conditions are favorable for deposition, rather than in the main stream. In fact, the main channel may even tend to enlarge, since the tendency is to fill the side channels and thus force the entire flow into the main channel. When the aggradation has progressed to the stage where the river can carry the entire sediment load brought to it by its tributaries, it seems probable that there will be a main channel largely free from islands, sloughs, lakes, ponds and secondary channels.

4.4 Summary

The major features of the drainage pattern in the Upper Mississippi basin 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 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, 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.

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 outwash, 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 x 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, 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 an analysis of the effects of development activities on the Upper Mississippi.

Although morphologic change within the channel itself is the norm, subsequent analysis shows that in terms of position within the floodplain the Upper Mississippi is 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 4-6). 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.

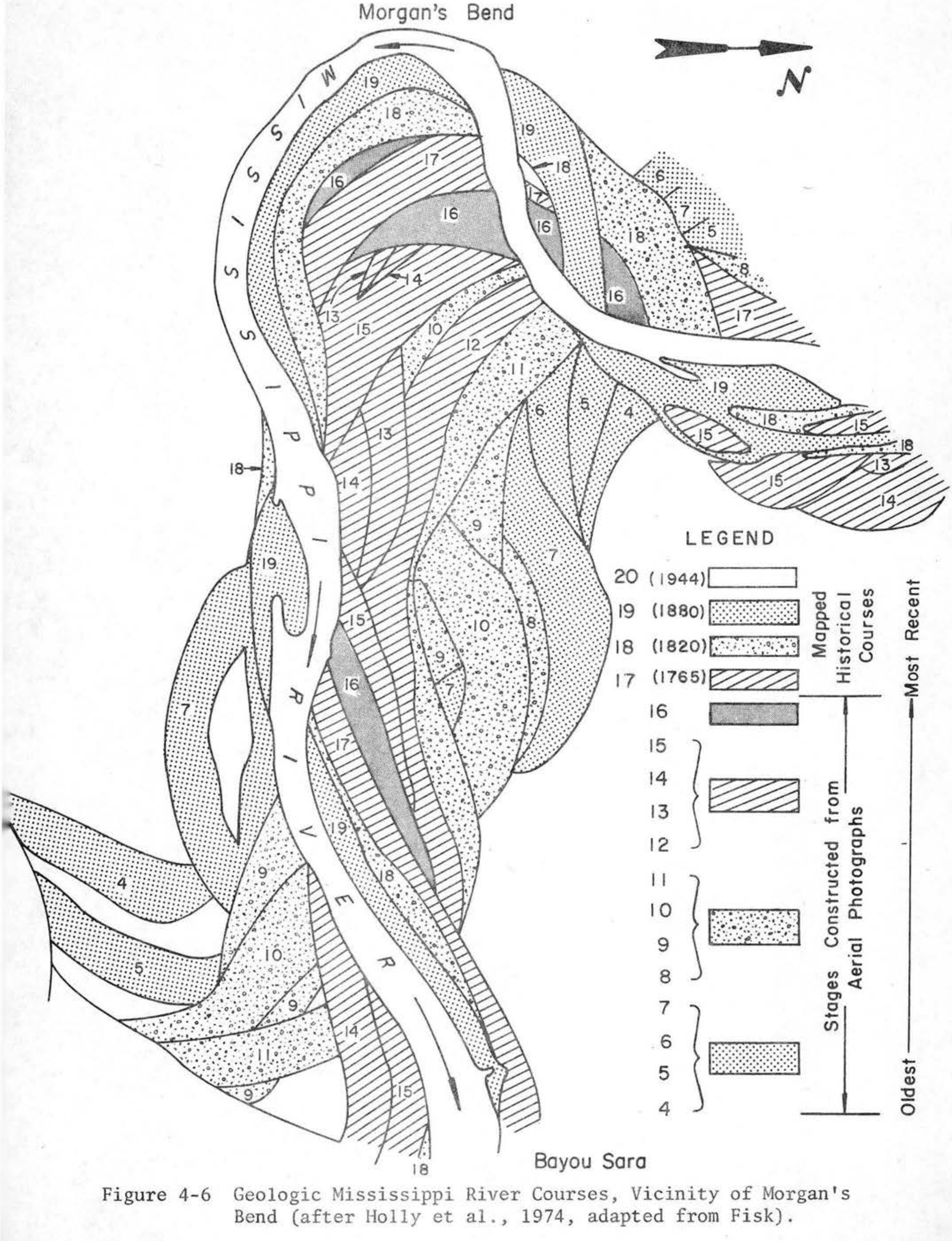


Figure 4-6 Geologic Mississippi River Courses, Vicinity of Morgan's Bend (after Holly et al., 1974, adapted from Fisk).

Chapter 5

RIVER RESPONSE

5.1 Introduction

The purpose of this chapter is to illustrate the application of the basic principles of hydraulics, hydrology, fluvial geomorphology and river mechanics to the analysis of river response. In general, the solution of complex problems in river engineering can be facilitated by a qualitative estimation followed by a quantitative analysis. For this reason, this chapter includes hypothetical cases of river environments and their response to a variety of natural and man-induced changes. The cases indicate qualitatively the trend of change in river morphology for given initial conditions. The analysis of these hypothetical cases is based on geomorphic and hydraulic principles presented in Chapters 2 and 3, and is intended to develop a general familiarity with the application of these principles to increasingly complex conditions of river response.

The quantitative analysis of river response problems is illustrated by applying river mechanics principles and the geomorphic approach to a specific reach of the Upper Mississippi River navigation system. The analysis in Section 5.3 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 encompasses Pools 24, 25, and 26 of the Upper Mississippi River navigation system.

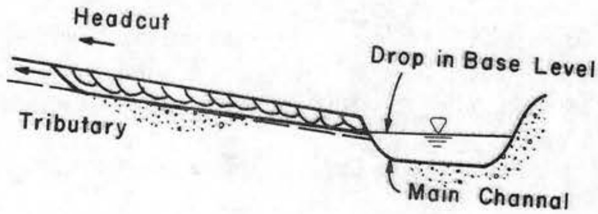
The Upper Mississippi River is viewed in the perspective of geologic time in Chapter 4. This survey of the geologic scene of the Upper Mississippi River basin leads to an understanding of the natural river prior to man's intervention which, in turn, provides the necessary baseline for an analysis of the response of the Upper Mississippi to such development activities as channel stabilization, construction of locks and dams, and dredging. Section 5.3 has a dual purpose in that it provides an illustration of the application of basic concepts of earlier chapters to a quantitative analysis of man's impact on a specific reach of the Upper Mississippi River; and it establishes the response of the lower pools of the Upper Mississippi River navigation system to the combined effects of contraction, dredging, and the construction of locks and dams. The response of this reach at the lower end of the navigation system is then available for comparison with the results of detailed studies of the upper pools of the navigation system.

5.2 Qualitative Analysis of River Response

To initiate analysis of the illustrative cases consider first several relatively simple situations commonly encountered in the river environment. Each case is introduced by a sketch which shows the physical situation prior to a selected natural or man-induced change. Below the sketch, some of the major local effects, upstream effects, and downstream effects resulting from natural processes or development activity are given. It is necessary to emphasize that only the gross local, upstream and downstream effects are identified. In analyzing river response it is worthwhile first of all to consider these gross effects. Having identified the qualitative response that can be anticipated, water and sediment routing techniques coupled with river mechanics relations introduced in Chapter 2, can be used to predict the possibility of change in river form and to estimate the magnitudes of local, upstream and downstream river response. This approach should be kept in view as one considers each of the cases outlined.

The initial river conditions are sometimes given in terms of storage dams, water diversions, etc. These examples are used as illustrations relating to common experience. In general, the effect of a storage reservoir is to cause a sudden increase of base level for the upstream section of the river. The result is aggradation of the channel upstream, degradation downstream and a modification of the downstream flow hydrograph. Similar changes in the channel result if the base level is raised by some other mechanism, say a tectonic uplift. The effect of diversions from rivers is to decrease the river discharge downstream of the diversion with or without an overall reduction of the sediment transport. Similarly, changes in water and sediment input to a river reach, often occur due to river development projects upstream from the reach under consideration or as a result of natural causes.

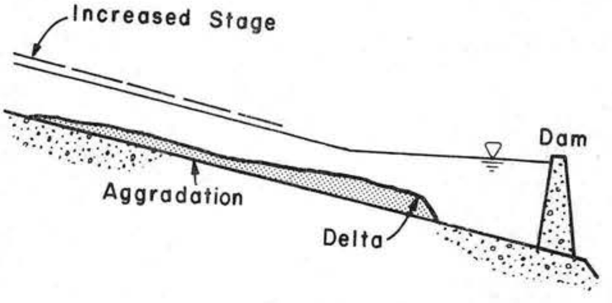
Figure 5-1 illustrates the confluence of a tributary stream with the main stem river. The average water surface elevation in the main channel acts as the base level for the tributary. It is assumed here that base level in the main channel has been lowered by a natural change in the river environment or by man-induced change such as the lowering of a reservoir level on the main stem. Applying Equation (3.9), $QS \propto Q_s D_{50}$, to the tributary stream it can be seen that the increase in slope S^+ must be balanced by an increase in sediment transport Q_s^+. Thus, under the new imposed condition, the local gradient of the tributary stream is significantly increased. This increased energy gradient induces head cutting and causes a significant increase in water velocities in the tributary stream. The result is bank instability, possible major changes in the geomorphic characteristics of the tributary stream and increased local scour.



Local Effects	Upstream Effects	Downstream Effects
1 - Headcutting	1 - Increased velocity	1 - Increased transport to main channel
2 - General scour	2 - Increased bed material transport	2 - Aggradation
3 - Local scour	3 - Unstable channel	3 - Increased flood stage
4 - Bank instability	4 - Possible change of form of river	4 - Possible change of form of river
5 - High velocities		

Figure 5-1 Lowering of Base Level for Tributary Stream.

Response to the converse situation, raising of base level, can be illustrated by considering river response to construction of a dam (Figure 5-2). Whenever the base level of a channel is raised a pool is created extending a considerable distance upstream depending on the amount of change. This "backwater" effect results in the M_1 curve of gradually varied flow (Figure 2-12). As the water and sediment being transported by the river encounters this pool, most of the sediments drop out forming a delta-like formation at the head of the pool which slowly advances downstream. The deposition of sediment at the entrance to the pool induces aggradation in the channel upstream. This aggradation may extend many miles upstream after a long period of time, producing significant changes in river geometry, and increased flood stages. Again, Equation (3.9) provides an indication of the response. The decrease in slope S^- must be accompanied by a decrease in transport capacity Q_s^- or $Q^oS^- \propto Q_s^- D_{50}^o$. In the extreme it is possible that the river may become sufficiently perched that at some high flow it could abandon the old channel and adopt a new one.



Local Effects	Upstream Effects	Downstream Effects
1 - Aggradation of bed	1 - See local effects	1 - See downstream effects, Figure 5-4
2 - Loss of waterway capacity	2 - Change in base level for tributaries	
3 - Change in river geometry	3 - Deposition in tributaries near confluences	
4 - Increased flood stage	4 - Aggradation causing a perched river channel to develop or changing the alignment of the main channel	

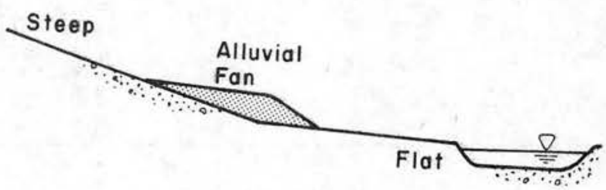
Figure 5-2 Raising Base Level in Main Channel.

As noted in Figure 5-2, the effects of raising the base level of the main channel include an increase in base level for any tributaries entering the pool formed by the main stem dam. The impact of this change on the tributaries is shown in Figure 5-3.

The change in gradient of the tributary stream in most cases causes significant deposition. This can be seen from Equation (3.9) where a decrease in slope is accompanied by a decrease in transport capacity:

$$Q^{o}S^{-} \propto Q_s^{-} D_{50}^{o}$$

assuming constant conditions of water discharge and size of bed material. In the case illustrated, an alluvial fan develops which in time can divert the river or reduce the waterway. In general, streams

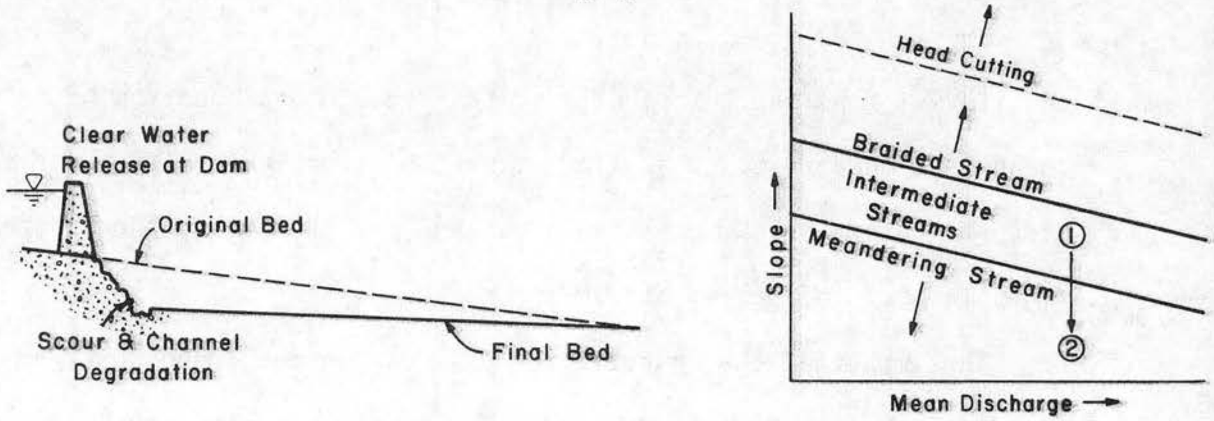


Local Effects	Upstream Effects	Downstream Effects
1 - Alluvial fan reduces waterway	1 - Aggradation	1 - Aggradation
2 - Channel location is uncertain	2 - Flooding	2 - Flooding
	3 - Possible change in river form and stability	3 - Development of tributary bar in the main channel

Figure 5-3 Raising Base Level for Tributary Stream.

on alluvial fans shift laterally so that the future location of the channel is uncertain. A similar situation occurs naturally where a steep tributary stream draining an upland region reaches the flatter floodplain of the parent stream.

The impact of the construction of a dam on the reach upstream of the dam is outlined in Figure 5-2. The response of the reach below the dam was discussed in Section 3.3.2 and is summarized briefly here. Construction of an upstream storage dam provides a desilting basin for the water flowing in the system (Figure 5-2). In most instances all of the bed-material load coming into a reservoir drops out within the reservoir. Water released from the reservoir is quite clear. The existing river channel is the result of its interaction with normal water-sediment flows over a long period of time. With the sediment-free flows the channel below the dam is too steep and sediments are entrained from the bed and the banks bringing about significant degradation. The channel banks may become unstable due to degradation and there is a possibility that the river, as its profile flattens, may change its plan form. A replot of Figure 3-7 is sketched in Figure 5-4 to illustrate the possible impact of a significant decrease in slope on channel pattern. Assuming that prior to dam construction the reach below the dam plotted as an intermediate stream (Point 1), the decrease in slope at constant water discharge could move the stream's plotting position to Point 2 in the meandering region of the chart. In the extreme case, it is possible that the degradation may cause failure of the dam and the release of a flood wave.



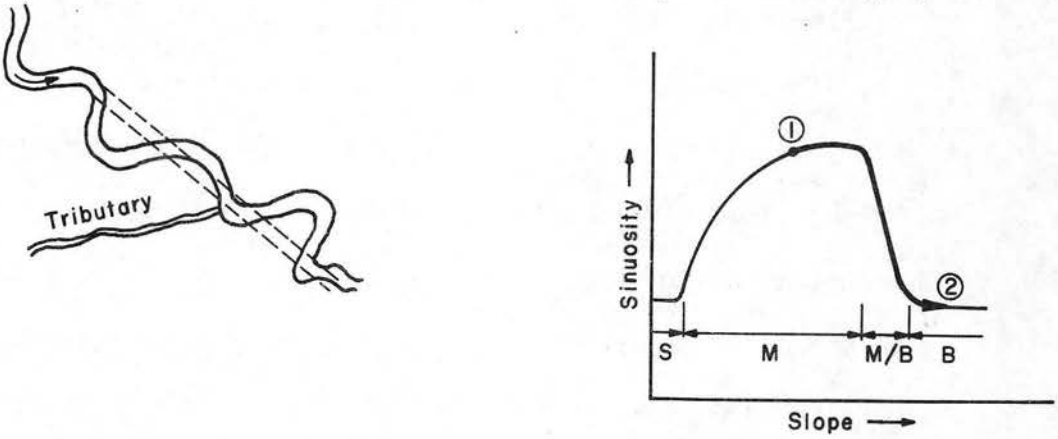
(a) (b)

Local Effects	Upstream Effects	Downstream Effects
1 - Channel degradation	1 - See upstream effects, Figure 5-2	1 - Degradation
2 - Possible change in river form		2 - Reduced flood stage
3 - Local scour		3 - Reduced base level for tributaries, increased velocity and reduced channel stability causing increased sediment transport to main channel
4 - Possible bank instability		
5 - Possible dam failure		

Figure 5-4 Clear Water Release below a Dam

Figure 5-5 illustrates a situation where artificial cutoffs have straightened the channel below a given reach. It is obvious that straightening the channel downstream of Reach A significantly increases the channel slope. In general, this causes higher velocities, increased bed material transport, degradation and possible head cutting through Reach A. This can result in unstable river banks and a braided streamform as shown on the sketch of Figure 3-6 included in Figure 5-5. Here, the original plotting position (Point 1) is moved to Point 2 in the braided region by the increase in channel slope. In addition, the straightening of the main channel brings about a drop in base level and any tributary streams flowing into the affected reach of the main channel are subjected to conditions outlined in Figure 5-1.

On the other hand, if the straightened section is designed to transport the sediment loads that the river is capable of carrying both



(a) (b)

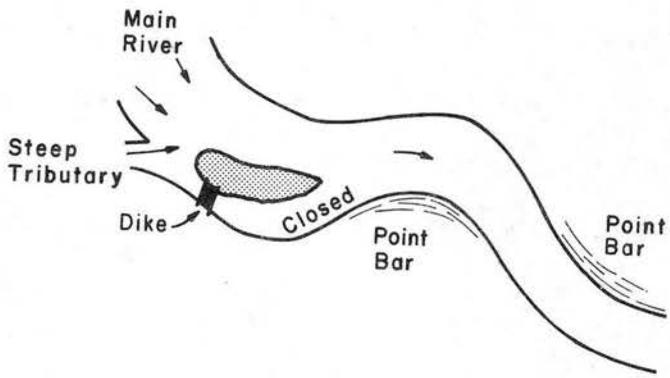
Local Effects	Upstream Effects	Downstream Effects
1 - Steeper slope	1 - See local effects	1 - Deposition downstream of straightened channel
2 - Higher velocity		2 - Increased flood stage
3 - Increased transport		3 - Loss of channel capacity
4 - Degradation and possible head-cutting		
5 - Banks unstable		
6 - River may braid		
7 - Degradation in tributary		

Figure 5-5 Straightening of a Reach by Cutoffs.

upstream and downstream of the straightened reach, bank stability may not be endangered. Such a channel should not undergo significant change over either short or long periods of time. It is possible to build modified reaches of main channels that do not introduce major adverse responses due to local steepening of the main channel. In order to design a straightened channel so that it behaves essentially as the natural channel in terms of velocities and magnitude of bed-material transport, it is necessary, in general, to build a wider, shallower section.

The development of an alluvial island below the confluence of a steep tributary is a common feature of the river environment (Figure 5-6). The tributary introduces relatively large quantities of bed material into the main channel. As a result of island formation in the main channel, divided flow exists. In an attempt to maintain navigation depths in the main channel it is common practice to close the chute channel by construction of a dike across the subchannel to the island or bar formed by deposition. Such a procedure forces all of the water and sediment to pass through a reduced width. This contraction of the river in general increases the local velocity, increases general and local scour, and may significantly increase bank instability.

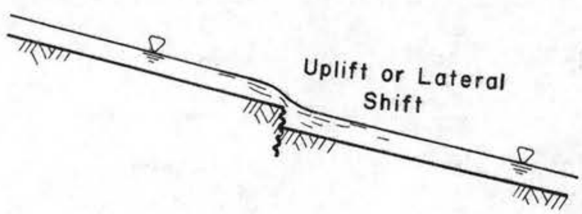
In addition, the contraction can change the alignment of the flow in the reach and thus would affect the downstream main channel for a considerable distance. A chute channel can develop across the next point bar downstream and its effect may extend several meander loops downstream. Upstream of the reach there is aggradation and its amount depends on the magnitude of water and sediment being introduced from the tributary. Also, there is significant increase in the backwater upstream of the reach at high flows which in turn affects other tributaries farther upstream. With this analysis the continuity equation (2.6), the relationship between discharge and bed-material transport (Figure 2-32), concepts of rapidly varied flow (Figure 2-19), and the backwater curves of Figure 2-13 all provide indicators of the response to be anticipated.



Local Effects	Upstream Effects	Downstream Effects
1 - Contraction of the river	1 - Aggradation	1 - Deposition of excess sediment eroded at and downstream of the closure
2 - Increased velocity	2 - Backwater at flood stage	2 - More severe attack at first bend downstream
3 - General and local scour	3 - Changed response of the tributary	3 - Possible development of a chute channel across the next point bar downstream
4 - Bank instability		

Figure 5-6 Closure of a Chute Channel.

Among the natural phenomena that can impact the river environment are earthquakes and tectonic activity (Figure 5-7). Large portions of the United States are subjected to at least infrequent earthquakes. Associated with earthquake activity are severe landslides, mud flows, uplifts in the terrain, and liquefaction of otherwise semi-stable materials, all of which can have a profound effect upon channels and structures located within the earthquake area. Historically, several rivers have completely changed their course as a consequence of earthquakes. For example, the Brahmaputra River in Bangladesh and India shifted its course laterally a distance of some 200 miles as a result of earthquakes that occurred approximately 200 years ago (Section 1.3.1). Although it may not be possible to design for this type of natural disaster, knowledge of the probability of its



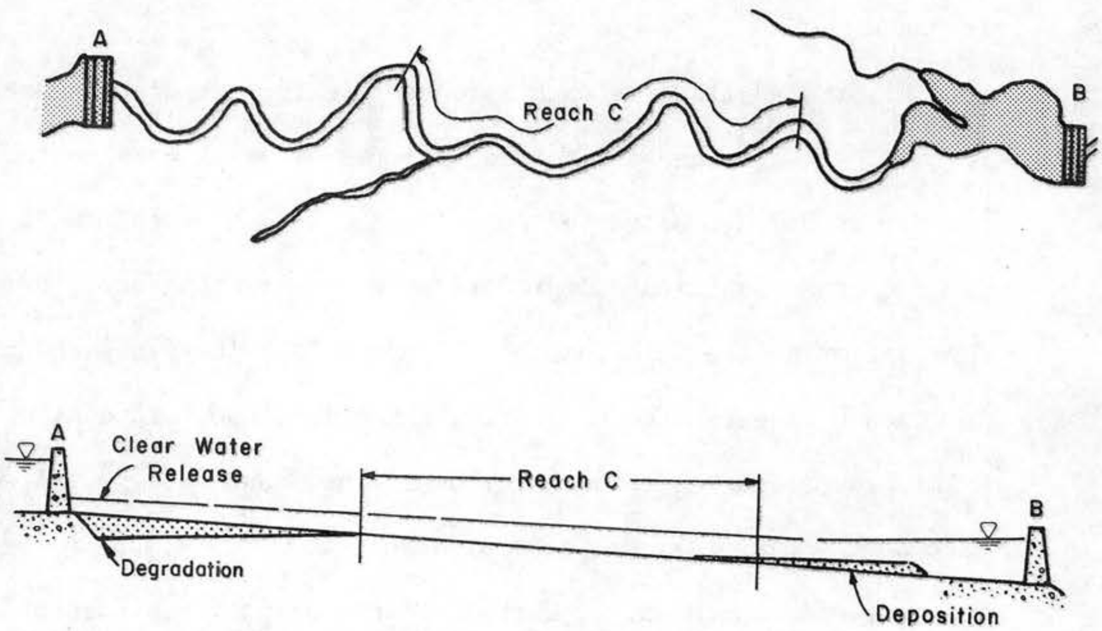
Local Effects	Upstream Effects	Downstream Effects
1 - Channel changes	1 - See local effects	1 - See local effects
2 - Scour or deposition	2 - Slide lakes	2 - Slide lakes
3 - Head cutting		
4 - Decrease in bank stability		
5 - Landslides		
6 - Rockslides		
7 - Mudflows		

Figure 5-7 Tectonic Activity.

occurrence is important so that certain aspects of the induced effects from earthquakes can be taken into consideration when evaluating river response.

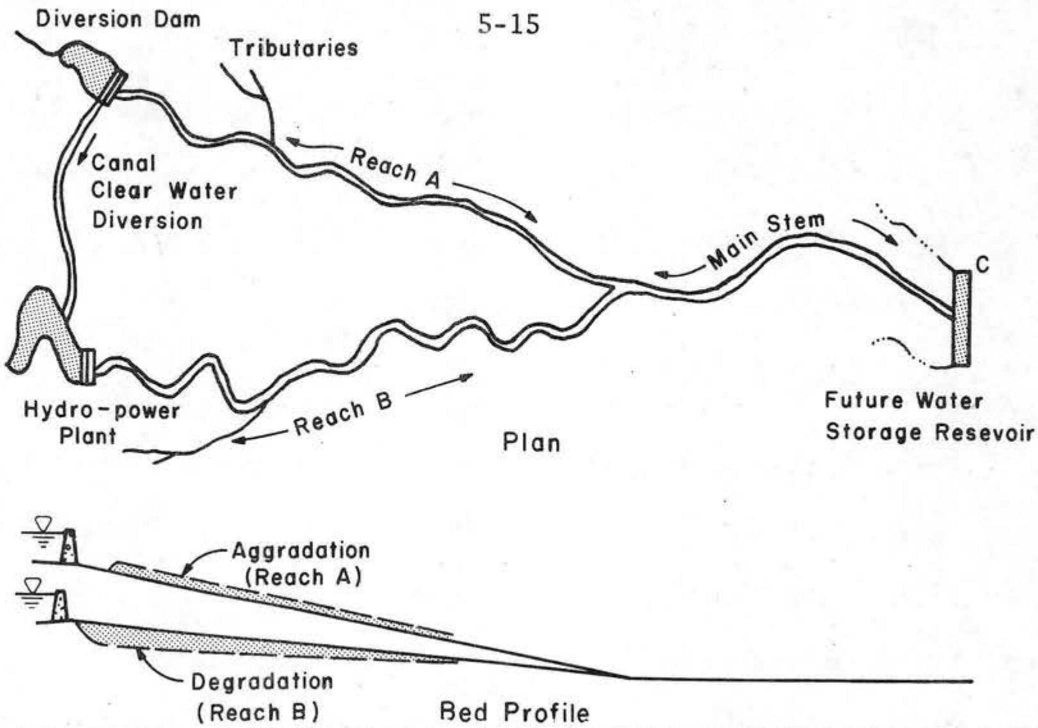
Figure 5-8 illustrates a more complicated set of circumstances. In this case a reach of river is affected by Dam A constructed upstream as well as Dam B constructed downstream. As documented in Figure 5-4, Dam A causes significant degradation in the main channel. Dam B causes aggradation in the main channel (Figure 5-2). The final condition in Reach C is estimated by summing the affects of both dams on the main channel and the tributary flows. The scour below Dam A would make some sedimentary material available for deposition in the reservoir above Dam B, further complicating the situation. Normally, this analysis requires water and sediment routing techniques studying both short- and long-term effects of the construction of these dams.

River response to upstream and downstream storage reservoirs on the same stream as analyzed in Figure 5-8 can be quite complex. Another situation that is amenable to a basic qualitative analysis involves the response of reaches on two major tributaries a considerable distance upstream of their confluence (Figure 5-9). Upstream of Reach A, a diversion structure is built to divert essentially clear water by canal to the adjacent tributary on which Reach B is located. Upstream of Reach B the clear water diverted from the other channel plus water from the tributary is released through a hydro-power plant. Ultimately, it is anticipated that a larger storage reservoir may be constructed downstream of the tributary confluence on the main stem at C. These changes in normal river flows



Local Effects	Upstream Effects	Downstream Effects
1 - Dam A causes degradation	1 - Channel could aggrade or degrade with effects similar to Figures 5-2 and 5-4	1 - See upstream effects
2 - Dam B causes aggradation		
3 - Final condition in Reach C is the combined effect of (1) and (2). Situation is complex and combined interaction of dams, main channel and tributaries must be analyzed using water and sediment routing techniques and geomorphic factors		

Figure 5-8 Combined Increase of Base Level and Reduction of Upstream Sediment Load.



Local Effects	Upstream Effects	Downstream Effects
1 - Reach A may be subjected to aggradation due to excess sediment left in the channel by diversion of clear water and degradation in tributaries caused by lowering of their base level	1 - Upstream of Reach A-aggradation and possible change of river form	1 - See upstream effects
2 - Reach B may be subjected to degradation due to increased discharge in the channel	2 - Upstream of Reach B-degradation and change of river form	2 - Construction of reservoir C could induce aggradation in the main channel and in the tributaries
3 - If a storage reservoir was constructed at C it could induce aggradation in both tributaries	3 - Channel instabilities	
	4 - Significant effects on flood stage	

Figure 5-9 Clear Water Diversion and Release Combined with Downstream Storage.

give rise to several complex responses in Reaches A and B on the tributary systems as well as on the main stem. Reach A may aggrade due to the excess of sediment left in that tributary when clear water is diverted:

$$Q^{-}S^{+} \propto Q_s^{o}\ D_{50}^{o}$$

However, initially there may be a lowering of the channel bed in the vicinity of the diversion structure because of the deposition upstream of the diversion dam and the release of essentially clear water for the relatively short period of time until the sediment storage capacity of the reservoir is satisfied. Reach B is subjected to degradation due to the increased discharge and an essentially clear water release:

$$Q^{+}S^{-} \propto Q_s^{-}\ D_{50}^{o}$$

However, the degradation of the channel could induce degradation in the tributaries causing them to provide additional sediment to the main channel. This response would to some degree counteract the degrading situation in this reach of river. Such changes in river systems are not uncommon and introduce complex responses throughout the system. Any complete analysis must consider the individual effects and sum them over time to determine a response in the reaches of concern.

The preceding illustrative examples have been analyzed to reveal qualitatively the anticipated geomorphic response to imposed natural or artificial change. The qualitative approach can also be applied to reveal the most probable modification in hydraulic parameters such as stage and discharge as a result of changes imposed on the river system. The plan view of a meandering channel as shown in Figure 5-10 exhibits significant changes in thalweg location between low-stage and high-stage flow. 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.

To prevent the development of chute channels and divided flow reaches it is common practice to construct dikes across the point bar as shown in Figure 5-10a. To protect the concave bank from erosion by low-stage flows revetment is usually placed opposite the point bar dike field. The impact of these river contraction works on the hydraulics of the bendway can be analyzed using concepts developed in Chapter 2. The change in roughness (Manning's n) with increasing discharge in a natural river is shown in Figure 2-27. This gradual decrease in roughness coefficient could be anticipated as conditions change from low-stage to high-stage flow in an undeveloped bendway and is sketched as the lower curve of Figure 5-10b. In the developed bendway, however, roughness conditions are altered significantly. As the discharge increases from low flow to high flow, the main thread of the flow impinges on the contraction dikes at an angle which approaches 90 degrees. The "shock" losses which result from turbulence generated by this impact as the flow is diverted around or spills over the dikes radically increase the roughness coefficient of the reach. Thus, for increasing discharge in the developed reach the trend in Manning's n can be sketched as the upper limb of Figure 5-10b. A partially developed river with, perhaps, a single dike instead of a dike field would produce an intermediate roughness curve as shown.

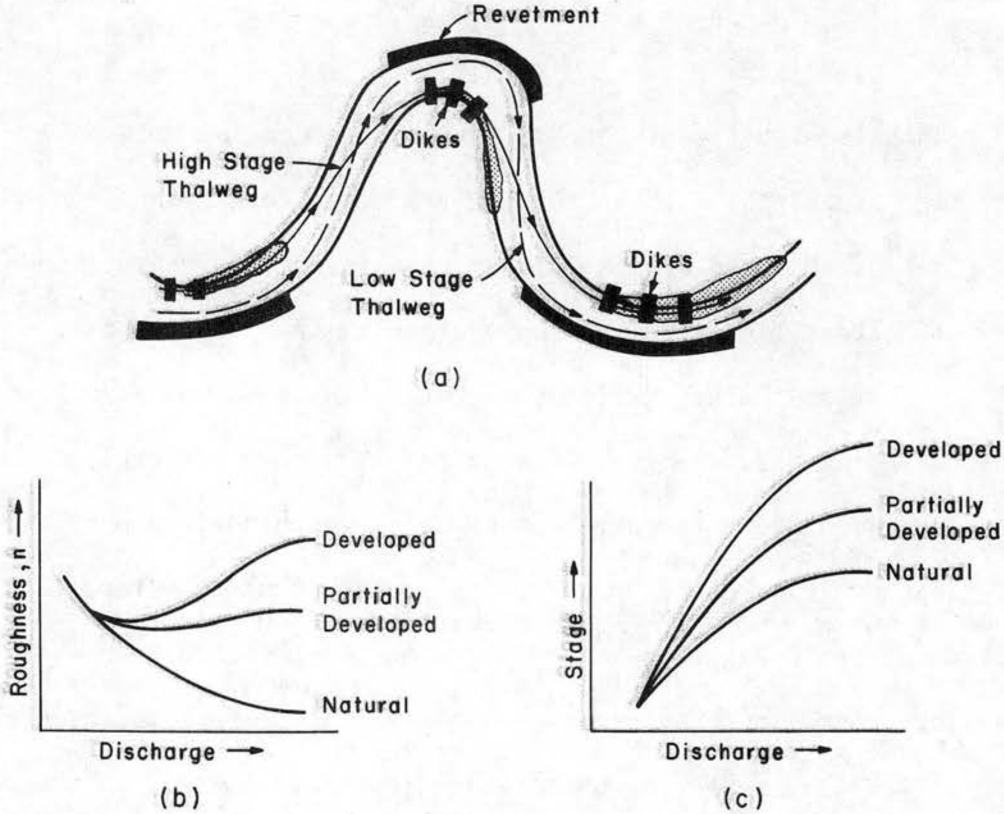


Figure 5-10 Change in Hydraulic Parameters in a Developed Reach.

The influence of this modification of the roughness characteristics of the reach as a result of constructing contraction works can be seen in Figure 5-10c. The stage discharge relation for the natural river appears as the lower curve of Figure 5-10c. The increased roughness of the developed reach reduces velocity (see Equation 2.31) and increases the stage, producing the modified stage-discharge relation approximated by the upper curve of Figure 5-10c. The intermediate stage-discharge curve reflects the diminished impact of partial development of the reach. The backwater resulting from increased stages would produce effects similar to those outlined in Figure 5-2 for a change in base level on the main stream.

To summarize the qualitative approach to the analysis of river response consider the problem of designing a river system to meet the conflicting demands of flood control and navigation. To minimize both flood damage and the height of levees required to provide flood protection it is desirable to reduce the stage associated with a given flood discharge. Conversely, to minimize navigation channel dredging requirements it is desirable to increase the stage associated with a given low flow. A qualitative analysis can point the way toward satisfying these apparently conflicting requirements.

In Figure 5-11a the low-stage and high-stage thalwegs in a typical meandering river system are illustrated. The low-stage thalweg (indicated by L) generally follows the outside or concave bank of the meander bends, thus traversing a relatively long flow path through the reach. Because of increased velocity and momentum, the high-stage flow line (indicated by H) generally short circuits the meander pattern, cutting across the tips of the point bars on the inside or convex bank of the meander bends, and following a shorter flow path through the reach. This reduced length of flow path for high-stage flows implies an increase in slope through the reach with effects similar to those associated with artificial cutoff of meander loops (Figure 5-5). The low-stage and high-stage characteristics of a river system designed to satisfy both flood protection and navigation requirements must be held within certain geomorphic and hydraulic limits. A qualitative analysis can assist in establishing these limits.

For example, the sinuosity of the low-flow river (plotted as L on Figure 5-11b) must be selected to insure that the increase in slope associated with high-stage flows does not plot in the braided region

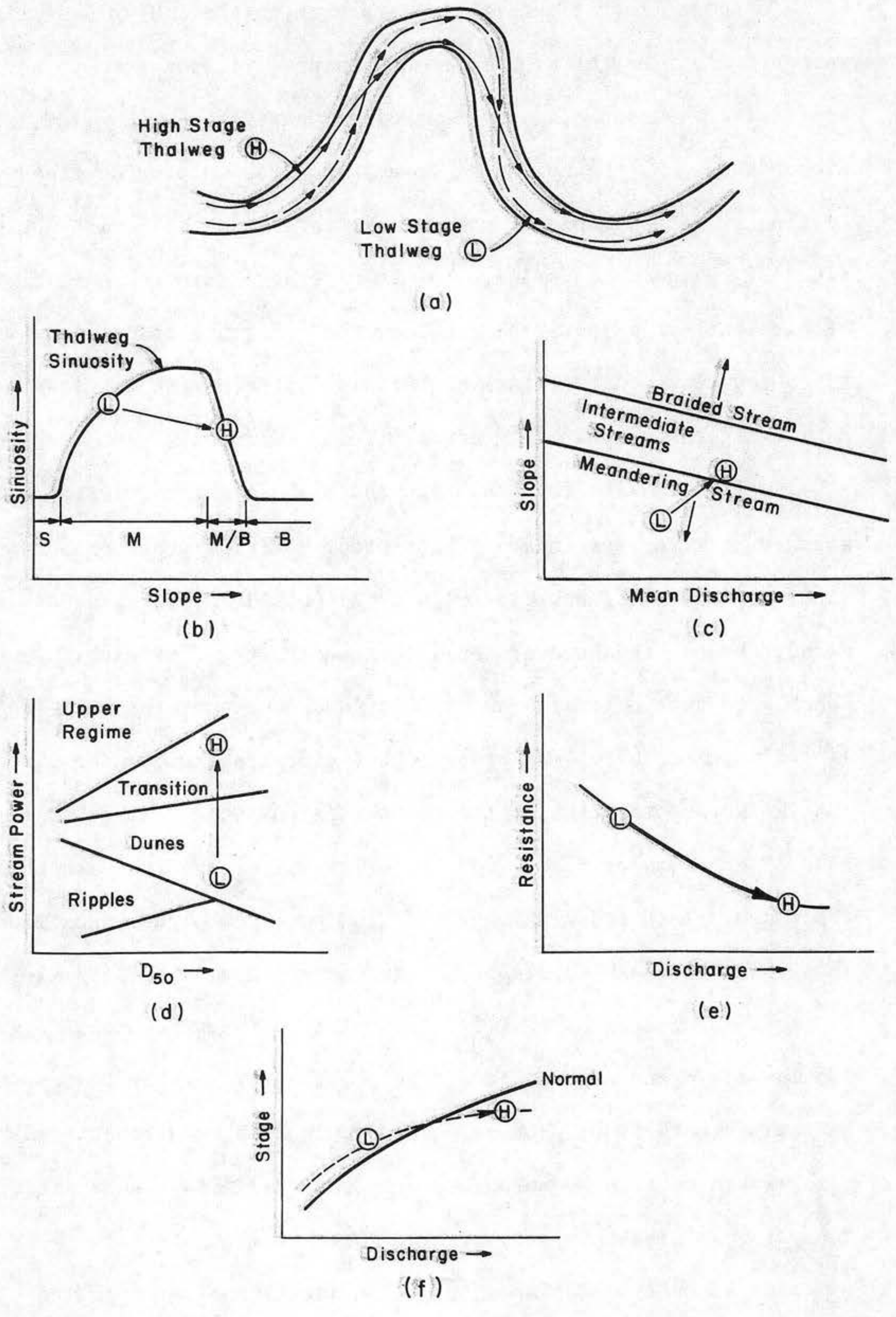


Figure 5-11 River System Design.

(H) Figure 5-11b). The limiting slope can be established on Figure 5-11c where (L) marks the desired plotting position well in the meander region under low-flow conditions, and (H) shows the upper limit under high-flow conditions. A plotting position located near the intermediate zone would be considered acceptable, however, the high-flow river should plot well below the braided region.

As indicated in Figure 2-23 the flow regime of an alluvial channel is closely related to stream power (τV) and, in turn, resistance to flow is a function of the type of bed form in the channel (Figures 2-24 and 2-27). Figure 2-23 is drawn schematically in Figure 5-11d. To provide a high resistance to flow and thus increase the stage associated with a given low-flow discharge, the low-flow river should plot within the dunes region of Figure 5-11d. The high-flow river should plot in the transition region but below upper regime conditions thus assuring a decreased resistance to flow and lower stages. The direct effect of controlling flow regime on the resistance coefficient of an alluvial river can be seen in Figure 5-11e. It should be recalled that development of a river radically alters the resistance function (Figure 5-10b) and greatly complicates the control of resistance to flow.

The end result of river system design to meet the conflicting demands of flood control and navigation is shown in Figure 5-11f. Here the normal stage-discharge relation (with constant resistance) is compared with stage-discharge plot for controlled low-flow and high-flow conditions. The low-flow stage discharge relation (L) has been achieved by controlling the flow regime to insure high resistance to flow and thus increased stages for a given low-flow discharge. The reduced stage for a given flood flow

H was obtained by increasing slope, velocity, and stream power to insure transition flow, decreased resistance, and decreased depth of flow. A careful control of slope and sinuosity was required to prevent a change of river form from the relatively easy to control meandering pattern to the less stable braided configuration. It must be emphasized that the plots used in a qualitative analysis such as this are applicable only to a specific river or river system. These plots must be developed from data derived from the particular river system of concern and qualitative relations used to supplement such an analysis must be checked with field data from the river in question.

5.3 Quantitative Analysis of River Response

5.3.1 Development of the Upper Mississippi River

The quantitative analysis of river response is illustrated by applying river mechanics principles and the geomorphic approach to a specific reach of the Upper Mississippi River navigation system. The analysis is concentrated in the Pool 24, 25, and 26 reach between Lock and Dam 22, just south of Hannibal, Missouri, and Lock and Dam 26 at Alton, Illinois (Figure 5-12). A necessary prelude to this analysis is a brief summary of development activity on the Upper Mississippi River.

5.3.1.1 Early Development. Man's activities on the Mississippi were mentioned briefly in Chapter 4 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

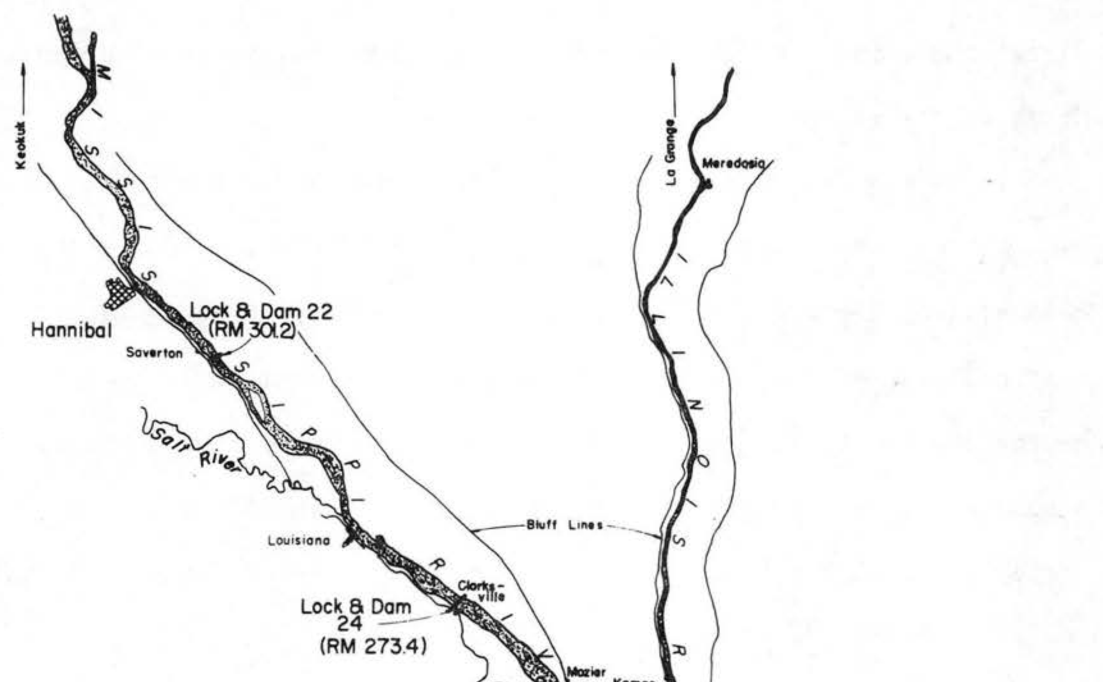


Figure 5-12 The Upper Mississippi River in the St. Louis District.

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 4.3.2). Between 1836 and 1840 Lieutenant R.E. Lee, Corps of Engineers, designed the first channel stabilization works on the Upper 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 and subsequent chapters.

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.

As an indicator of the extent of dike construction under the $4\frac{1}{2}$- and 6-foot projects, the locations of the 83 dikes that have been constructed in Pool 24 are shown in Figure 5-13. 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.

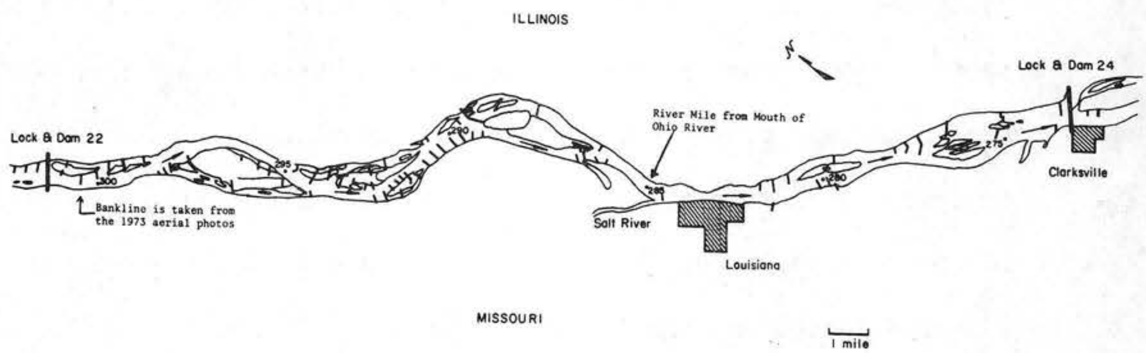


Figure 5-13 Location of Dikes in Pool 24, Upper Mississippi River.

For more than a century, levees have been used in the Upper Mississippi River basin to protect the people and floodplain property from floods. By 1891, there were more than 40 miles of levee along the Upper Mississippi River bank in the Pool 24, 25, 26 reach. The longest levee in 1891 was the SNY levee that extended along the Illinois side of the Mississippi River from just above Mosier Landing (RM 260.0) upstream past the location of Lock and Dam 22 (RM 301.2). Other levees were not so extensive. On the Missouri floodplain in the Pool 25 reach some individual fields were protected by levees. Otherwise, the Missouri floodplain in the study reach was mostly unprotected.

The effects of the levees on a river system are usually twofold. First, sedimentation on the floodplain is arrested because the floodplain is inundated only when the levees fail or are overtopped. Second, the storage capacity of the floodplain is no longer available to help attenuate flood peaks.

By 1929, improvements had been made to the SNY levees protecting the Illinois floodplain from the Mississippi River in the Pool 24 and 25 reach. On the Missouri side, levees were built along the bankline from Bobs Creek (RM 239) up along the Pool 25 reach of river to the bluff line near Clarksville, Missouri (Figure 5-12). Also, a small section of Missouri floodplain immediately upstream of the Salt River confluence was protected. In the Pool 26 reach, there were a few miles of levees back from the river near Dardenne Creek and the Cuivre River. The present day levee system between Hannibal and St. Louis is essentially the same as the 1929 system.

5.3.1.2 The Nine-Foot Channel Project. 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 great 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 completely regulate the flow, as well as supplemental dredging to maintain the channel.

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 form a series of badly polluted pools that 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."

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, Iowa 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 reestablishment of the aquatic and marsh vegetation, the refuge should be an improved place for waterfowl and probably also for muskrats."

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."

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 5-14. 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 5-14 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.

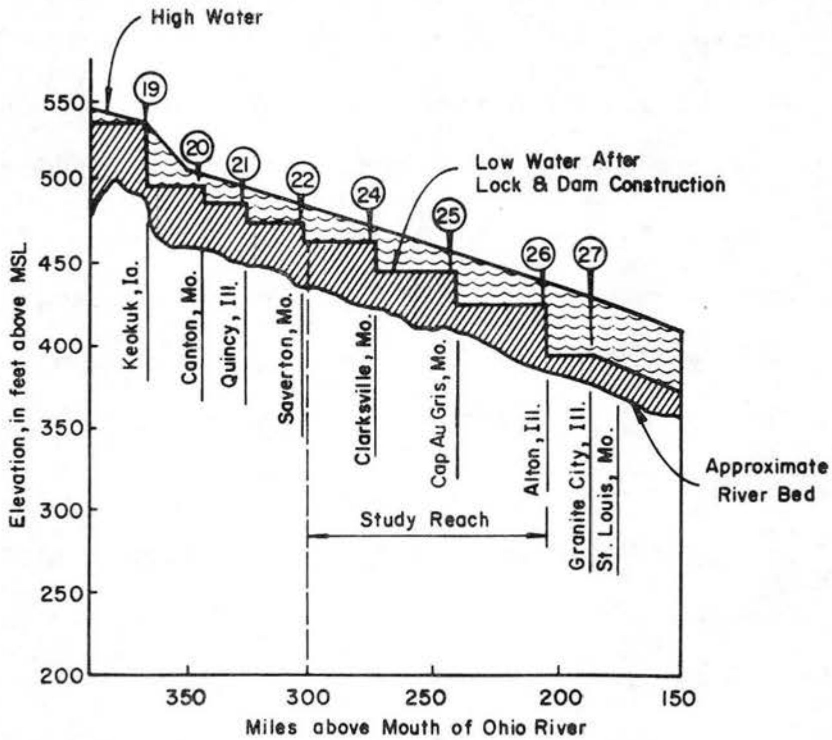


Figure 5-14 The Navigation Stairway (from a pamphlet titled "The Upper Mississippi River...Nine-foot Channel," U.S. Army Corps of Engineers)

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 flood periods the gates are lifted entirely above the water level. The majority of lock chambers for the 9-foot channel project are 110 feet wide and 600 feet long. In addition to the locks 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 many 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 include 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 is 80 feet wide and 25 feet high.

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.

5.3.2 Geomorphic Response of the Upper Mississippi River

5.3.2.1 River Position. The position of the Upper Mississippi in the Pool 24, 25, and 26 study reach was sketched briefly in Section 3.5.2 in regard to tributary influence on alluvial island formation. From Hannibal to Clarksville (Figure 5-12) 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 River 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 can be 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 5-16 through 5-19 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 5-18 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 in this reach.

5.3.2.2 River Width. For consistency in any geomorphic study definitions for basic parameters must be established and adhered to. Here 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 (Figure 5-12) 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 5-1). Examination of maps and aerial

Table 5-1 Average River Surface Widths*

Pool 24				
	Average Width, ft			
Location	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 1891 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.

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 5-1 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 Lock and Dam 24 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 (1927-1973) 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 5-19), major width changes occurred after construction of a 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 5-18) 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 24 can be seen in Table 5-1 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.

5.3.2.3 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 5-15. Because the lengths of the subreaches in Pool 24 did not change in the last 100 years, the change in surface areas has mirrored the change in river width. Thus, the surface area in the middle reach of 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 5-15). 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.

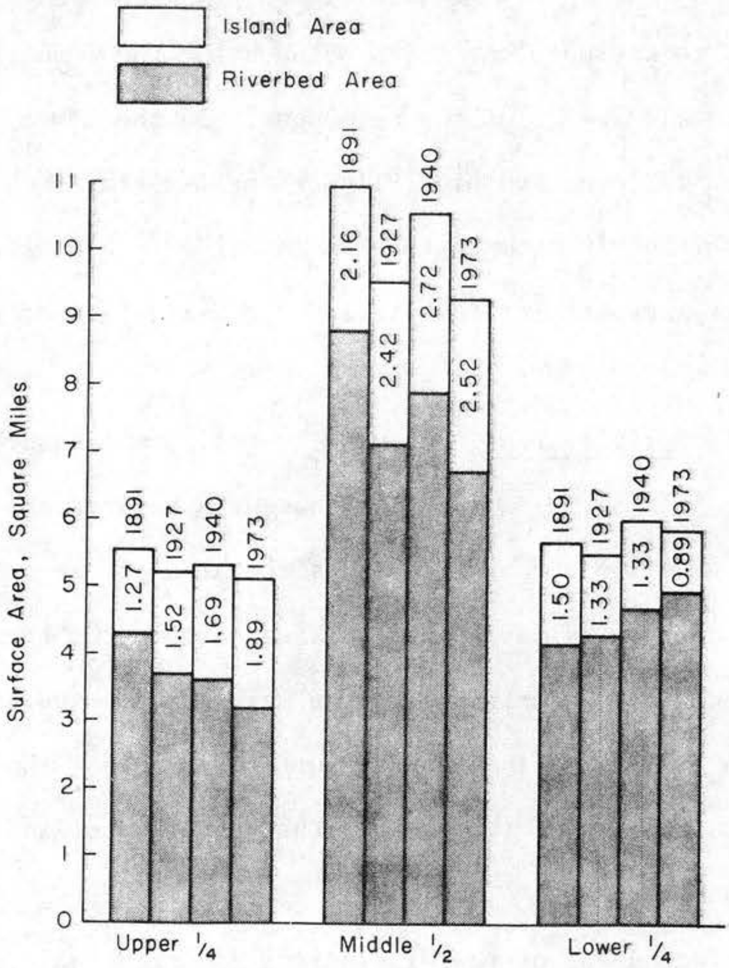


Figure 5-15 Pool 24 Surface Areas.

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 5-16 through 5-19. 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 5-16) provide an excellent example of decrease in island area

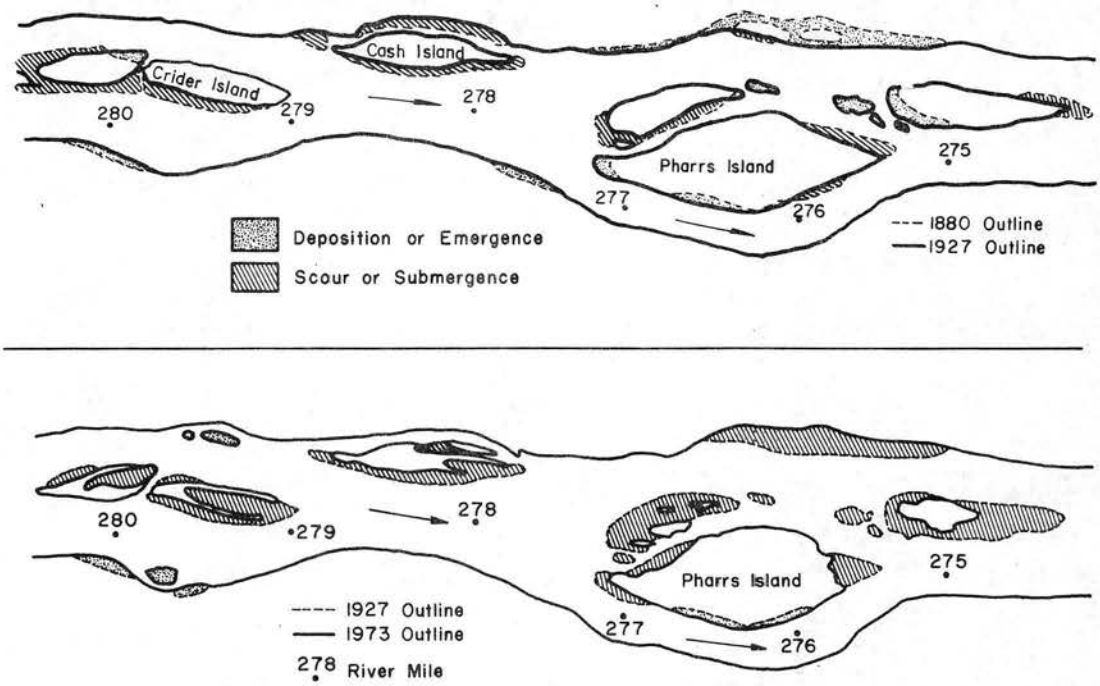


Figure 5-16 Crider-Cash-Pharrs Island Area.

in the lower 1/4 of a pool, and incidentally, illustrate the crab-claw shape so indicative of the processes of island building (Section 3.5.2).

In the Clarksville-Carrol Island area immediately below Lock and Dam 24 (Figure 5-17) 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.

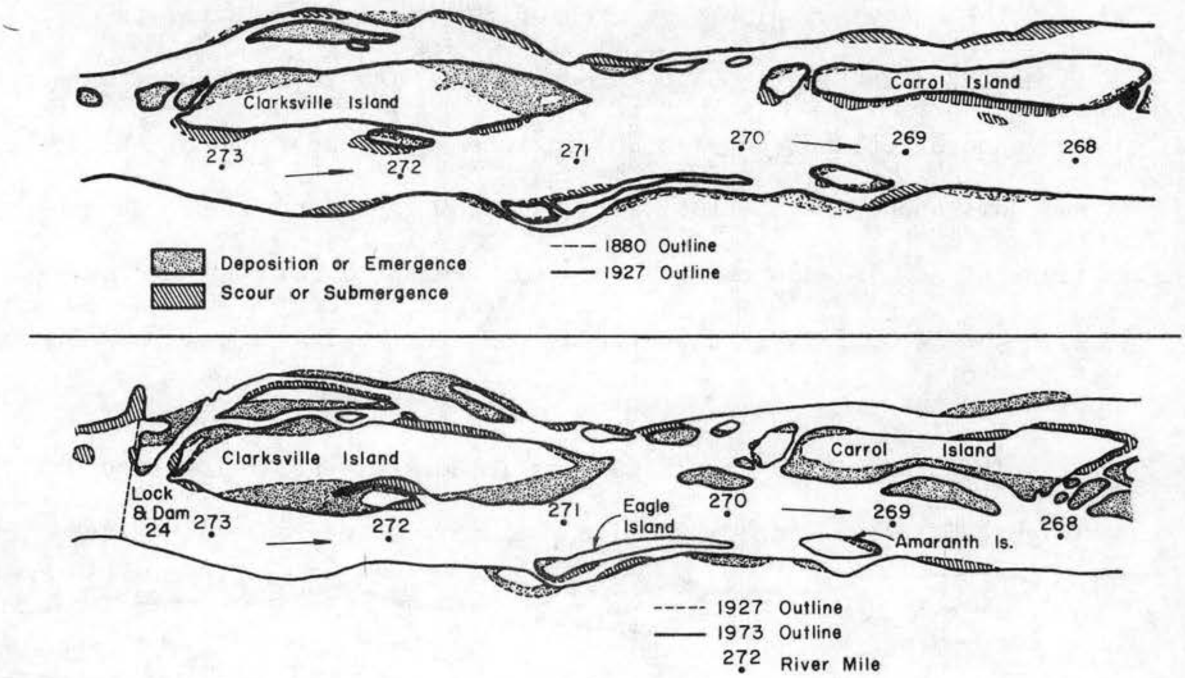


Figure 5-17 Clarksville-Carrol Island Area.

The Mosier Island reach in the middle half of Pool 25 (Figure 5-18) shows the growth of several major islands (1880-1927) just upstream of

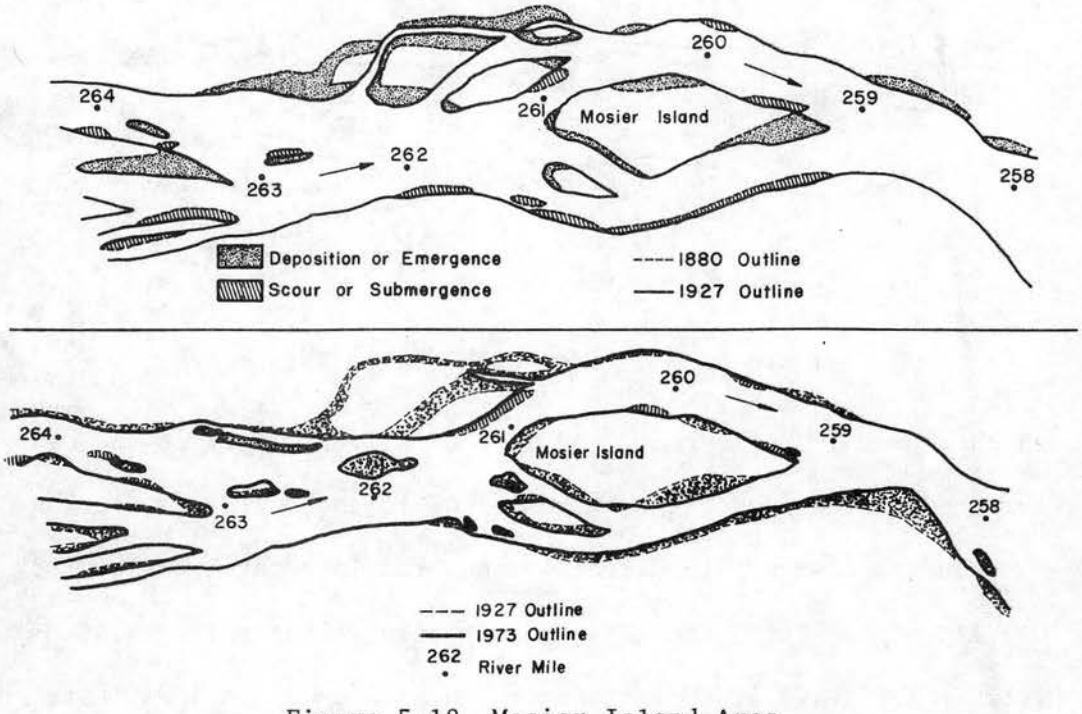


Figure 5-18 Mosier Island Area.

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 5-19) 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

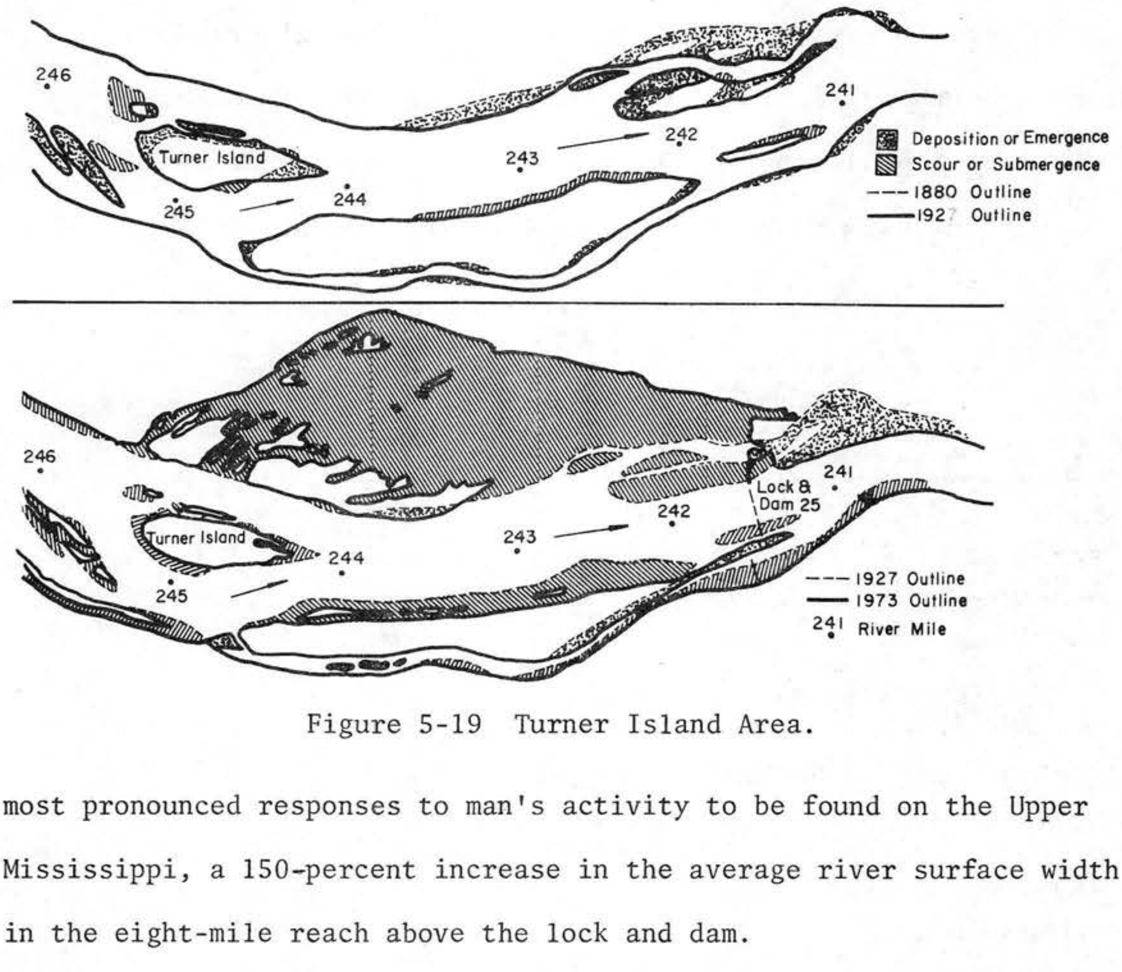


Figure 5-19 Turner Island Area.

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 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.

5.3.2.4 The Longitudinal Profile. The longitudinal profile of the Upper Mississippi from Keokuk, Iowa to St. Louis is shown in Figure 5-14 together with the low-water surface before and after construction of the locks and dams. In the Pool 24, 25, and 26 reach, 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 5-2. Also changes in bed elevation in the thalweg are included in Table 5-2 for

Table 5-2 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

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. 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 5-20.

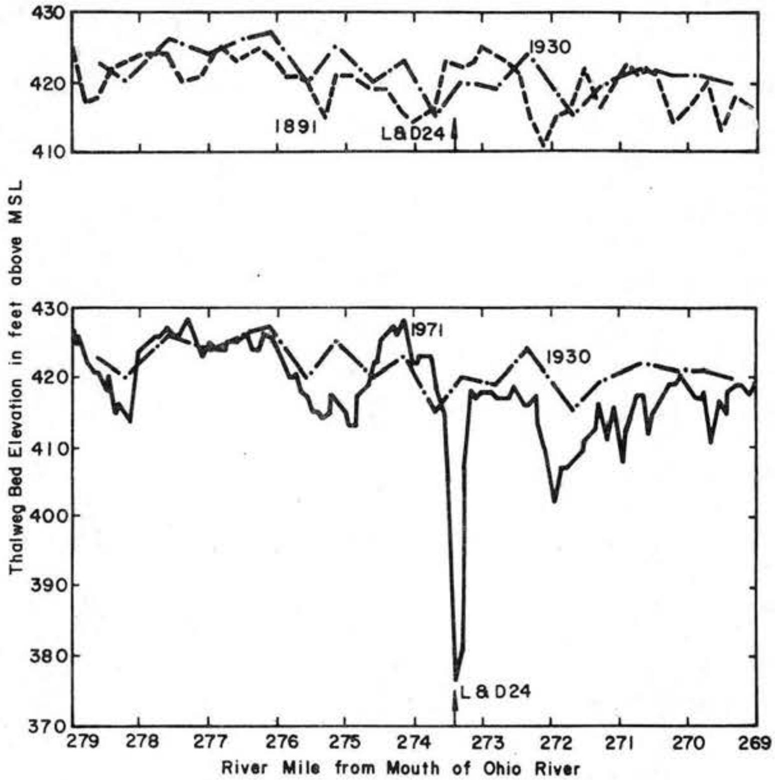


Figure 5-20 Longitudinal Profiles through Lock and Dam 24.

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 Equation (3.9) (see Figures 5-2 and 5-4).

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. Lane's description of the Upper Mississippi as more braided in character than meandering has been cited (Section 4.3.3). Further, Lane 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 bankfull stage and apparently exercised a major influence on the flow at only the lower stages. For example, prior to 1891, 41 dikes were constructed in the Pool 24, 25, and 26 reach of the Upper Mississippi River, 15 of them in what is now Pool 25. These dikes were all low structures 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 riverbed in the deep part of the channel has aggraded approximately two feet since construction of Lock and Dam 24.

5.3.2.5 The River in Cross Section. 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 5-21, 5-22, and 5-23 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 5-3.

In Pool 24, the Cottel Island section (Figure 5-21) 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 5-13--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

Table 5-3 Cross-Sectional 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

(RM 298). The sequence of aggradation between 1891 and 1930 followed by degradation is reflected in the average depth in Table 5-3 and can be observed in the main channel in Figure 5-21. The marginal effectiveness of the early dikes in this reach as well as significant contraction

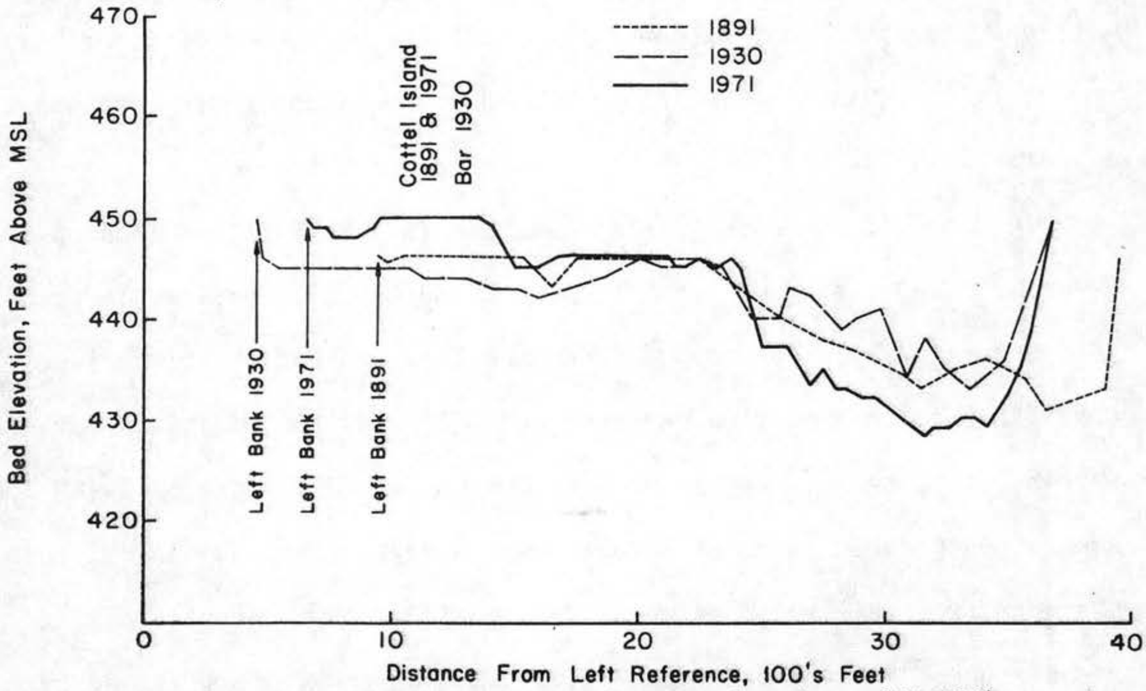


Figure 5-21 Cottel Island Cross Section (RM 300).

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 7.

The Eagle Island reach is located at River Mile 270 in Pool 25, 3.5 miles downstream of Lock and Dam 24 (Figure 5-22). Dike construction along the west bank of Clarksville Island (Figure 5-17) 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 5-17) 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 5-3) 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

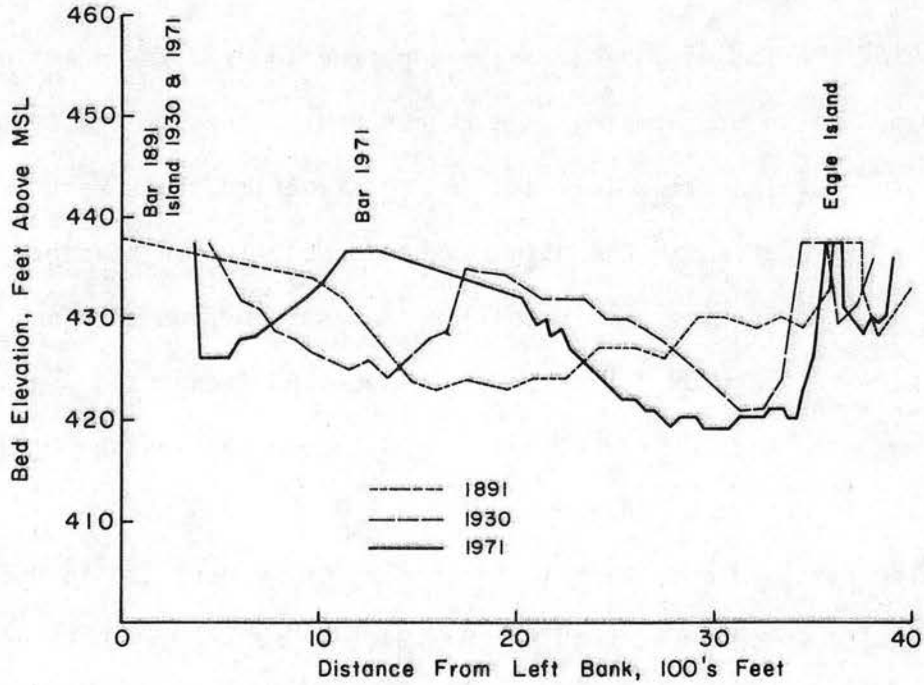


Figure 5-22 Eagle Island Cross Section (RM 270).

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 7.

In Pool 26, the Turkey Island reach (Figure 5-23) is located adjacent to Cuivre Island (RM 237) and the Cuivre River. This reach

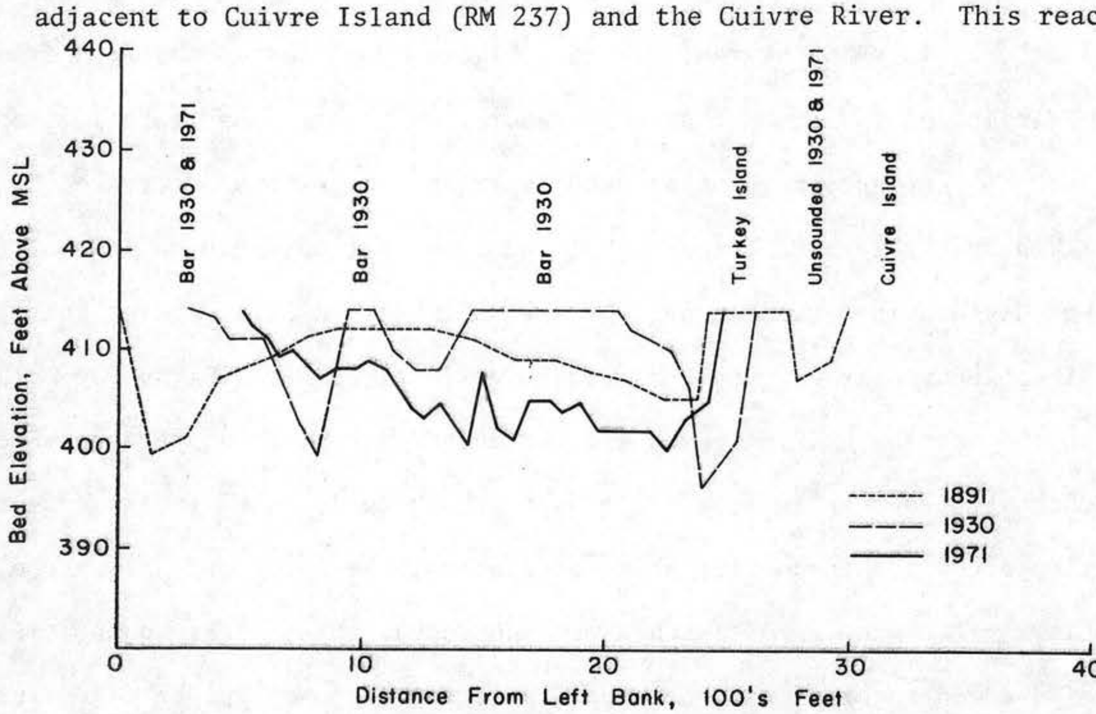


Figure 5-23 Turkey Island Cross Section (RM 236.9).

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 performed 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 5-3) 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.

5.3.3 Hydraulic Response of the Upper Mississippi River

In the Pool 24, 25, and 26 reach 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 also reflected in trends in the stage and discharge record.

5.3.3.1 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 5-12) 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 4-1), 65 river miles above the 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 5-12). 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.

5.3.3.2 Discharge and Stage Trends. The annual maximum, mean and minimum discharges for Alton and Keokuk are given in Figures 5-24 and 5-25. The annual maximum and minimum stages at Alton, Keokuk, Hannibal and Grafton are shown in Figures 5-26 through 5-29 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 5-4. 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

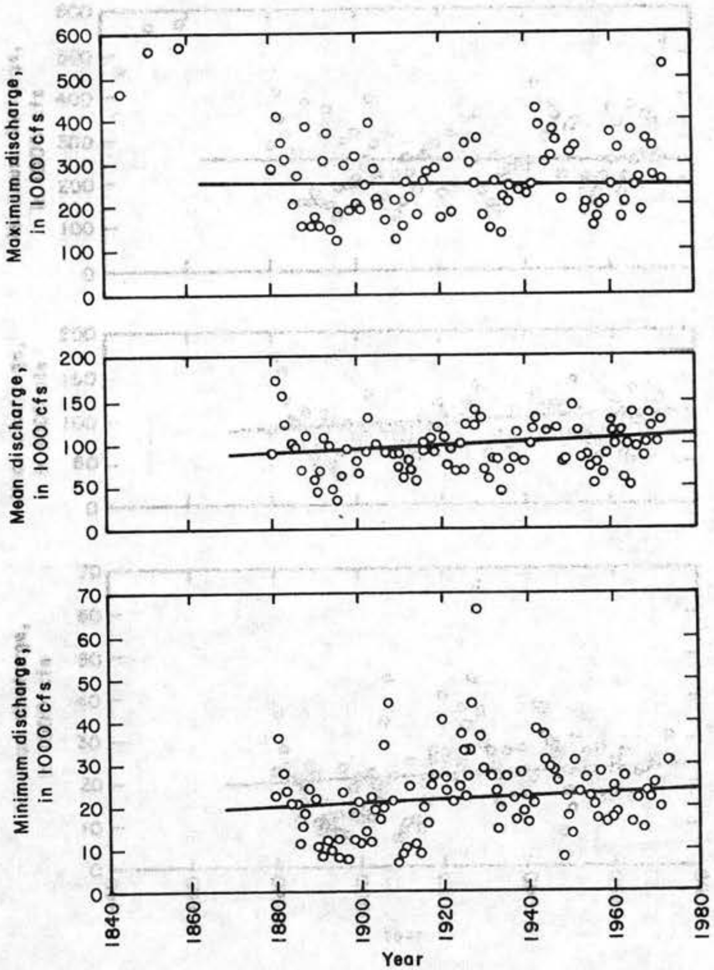


Figure 5-24 Annual Discharges at Alton.

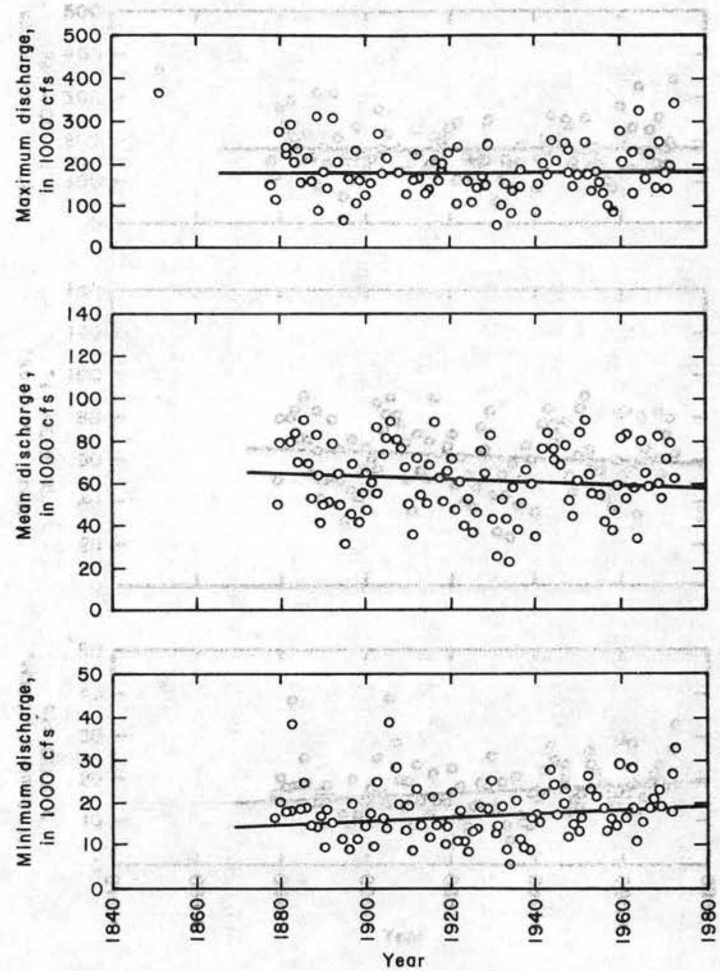


Figure 5-25 Annual Discharges at Keokuk.

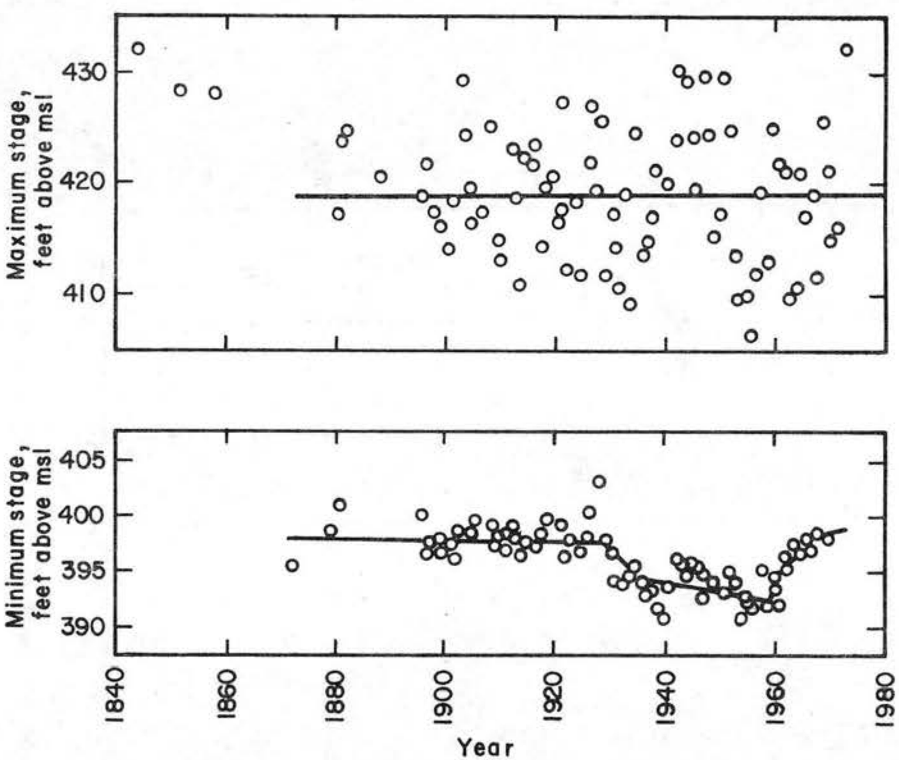


Figure 5-26 Annual Stages at Alton.

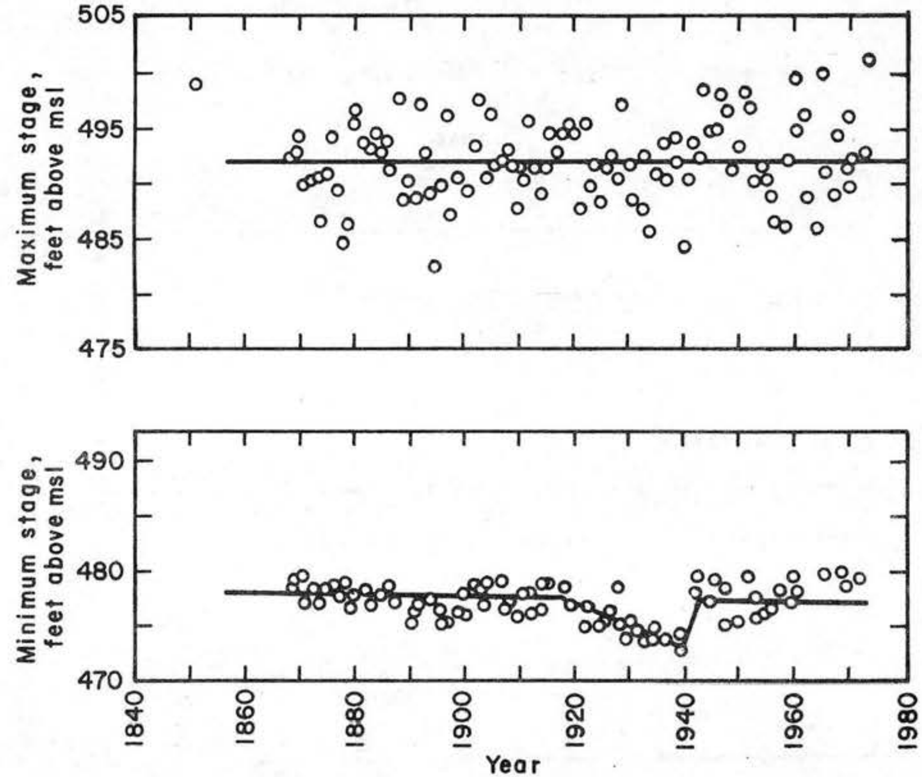
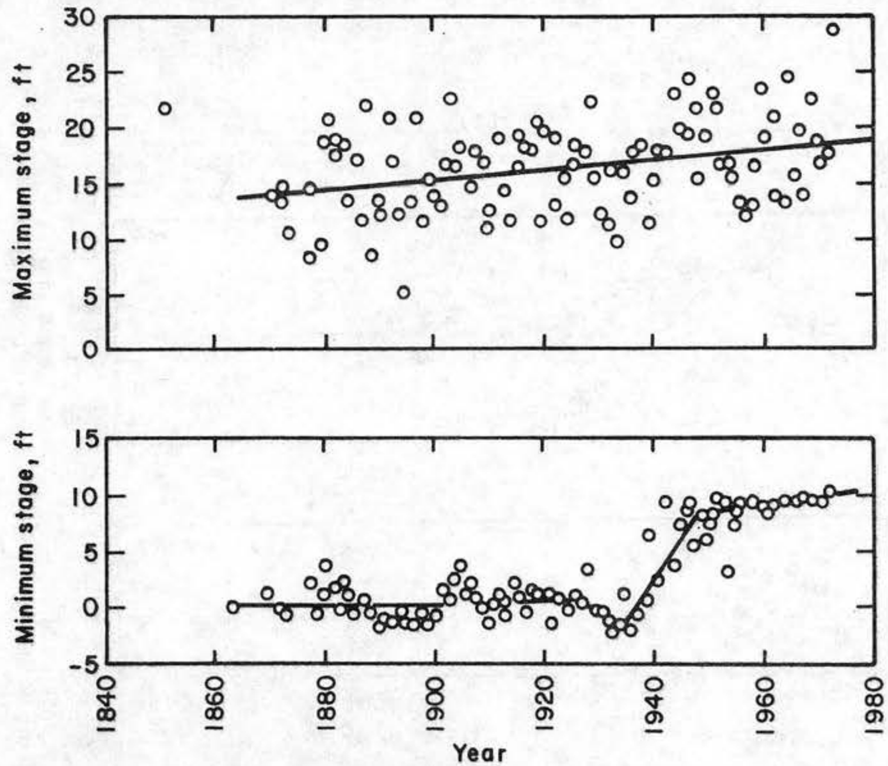
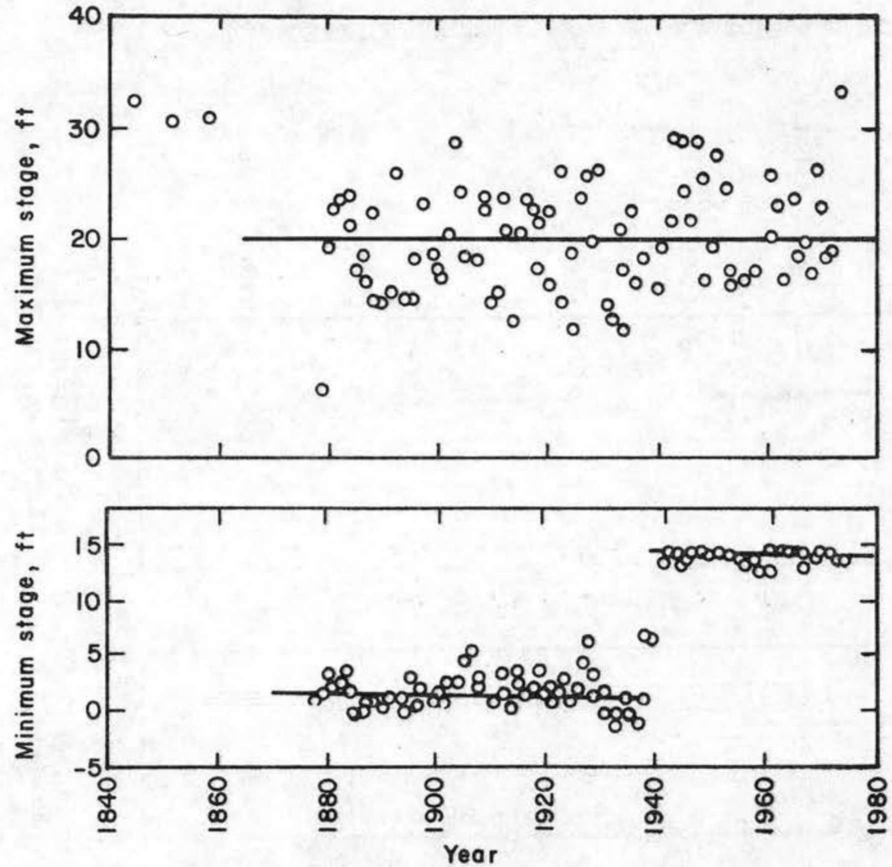


Figure 5-27 Annual Stages at Keokuk.



Note: Zero on the Hannibal gage is at 449.07 ft above msl.

Figure 5-28 Annual Stages at Hannibal.



Note: Zero on the Grafton gage is 403.79 ft above msl.

Figure 5-29 Annual Stages at Grafton.

Table 5-4 Trends in Annual Discharges and Stages

Location	Maximum Stage	Minimum Stage	Maximum Discharge	Average Discharge	Minimum Discharge
Mississippi River at Alton	None	Down & then up	None	Up	Up
Mississippi River at Keokuk	None	Down & then up	None	Down	Up
Mississippi River at Hannibal	Up	Up			
Mississippi River at Grafton	None	Up			

the immediate increase in minimum stage is apparent in Figure 5-29. This same response can be seen at Keokuk and at Hannibal in the late 1930's following construction of Locks and Dams 20 (Figure 5-27) and 22 (Figure 5-28) respectively, and at Alton (Figure 5-26) in the early 1960's when Dam 27 below St. Louis was completed. The minimum stage records at both Alton and Keokuk (Figures 5-26 and 5-27) show a definite decrease following lock and dam construction in 1914 and 1938 respectively till the early 1960's and the late 1930's, which can be attributed to degradation immediately below each lock and dam.

5.3.3.3 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 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 5-5 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 2-3).

A significant increase in sediment yield from the northern portions of the Upper Mississippi basin to the southern portions of the basin has been documented in the Upper Mississippi River Comprehensive Basin Study (1972). More specifically, from Wabasha, Minnesota (Figure 4-1) 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,

Table 5-5 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)

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 (Figure 4-1). 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.

5.3.4 Summary and Contrasts

The analysis of the geomorphic and hydraulic response of the Pool 24, 25, and 26 reach of the Upper Mississippi has been presented to illustrate the application of river mechanics principles and the geomorphic approach. River response revealed by this analysis was the result of the combined effects of contraction, dredging, and the construction of locks and dams, and no attempt was made to isolate the response to an individual development activity. In Chapters 6 and 7, respectively, the response to the environmentally critical activities of contraction works and dredging will be analyzed individually. The response of the Pool 24, 25, and 26 reach to the combined activities of contraction, dredging, and construction of locks and dams is summarized in this section and compared briefly with the results of a similar analysis of the response of the Middle Mississippi River (Figure 4-1) to man's activity conducted by Simons, Schumm, and Stevens (1974).

This comparison contrasts the response of a reach subjected to flow regulation by navigation dams with the response of a reach developed by open river training works (contraction dikes and revetment).

Development for navigation on both the Upper and Middle Mississippi Rivers between 1890 and 1930 followed the same theme of contraction, revetment, and dredging where necessary to attain desired depth. After 1930 the Corps of Engineers 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 was 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 selected by the Corps of Engineers for detailed study, the riverbed 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 lower three pools of the navigation system 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 on the Middle Mississippi 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 River. 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. In general 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. On the average, the annual maximum stages and discharges at Alton and Keokuk have remained unchanged in the last 100 years. This indicates that present day floods on the upper river produce flood stages similar to the past.

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.

5.4 Mathematical Modeling of River Response

The basic concepts of a one-dimensional mathematical model for water and sediment routing in natural channels are described in detail in Section 2.4.4. The Pool 24, 25, and 26 reach was used to illustrate the application of this model to a specific reach of river (Figure 2-35), and the development of the model through the calibration stage (Figure 2-37) was outlined. With the calibration process complete, the model can be employed to study the river's response to alternate operating schemes and future development.

While the quantitative analysis of Section 5.3 delineates river response to past development activity, such an analysis provides an indication of future response only through inference and causal reasoning. The calibrated mathematical model, however, permits an analysis of future geomorphic change. What will Pools 24, 25, and 26 in the Upper Mississippi and Lower Illinois Rivers look like 50 years from now? Will the side channels fill with sediment? Will the river-bed aggrade making the maintenance of the 9-foot channel project more expensive? Are there any viable alternatives to the present-day operations that would enhance the environmental aspects of the pools and at the same time maintain the navigation channel? The application of a mathematical model to answer these and other questions relative to the Pool 24, 25, and 26 reach is described in this section.

5.4.1 Model Operation

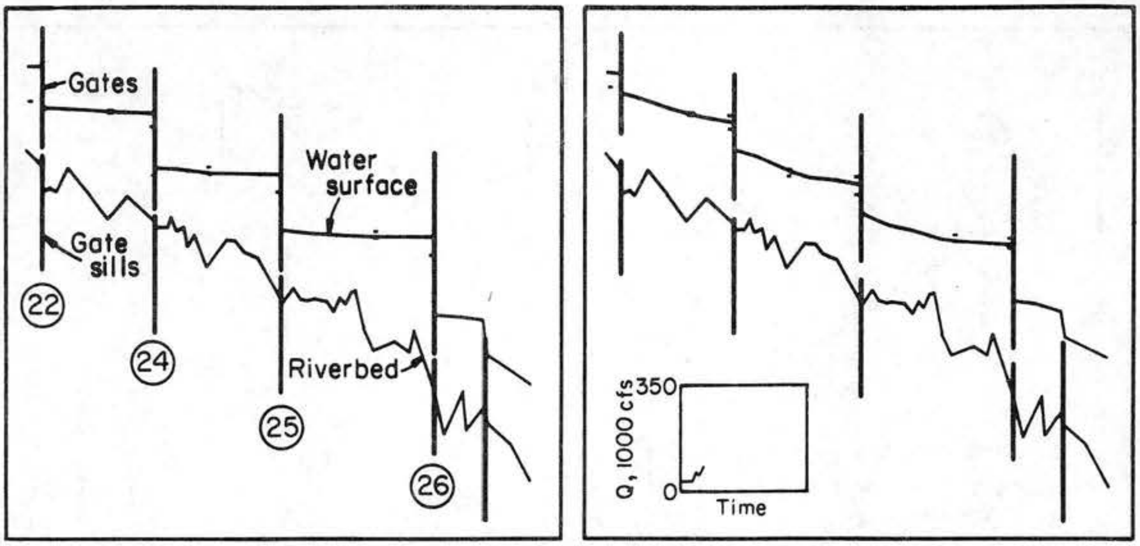
To illustrate the general operating characteristics of the mathematical model consider, first, typical results of routing a flood through the Pool 24, 25, and 26 reach as shown in Figures 5-30 and 5-31.

The water surface profile in the Upper Mississippi River for a discharge (Q) of 28,000 cfs at Day 24 is shown in Figure 5-30a. To maintain the normal pool levels, the control gates are lowered close to the gate sills. As the inflow increases, the pool stage is lowered at the dam by gradually opening the gates to maintain the level at the control stations within the prescribed control limits as shown in Figure 5-30b (pool operations and pool control points are described in Section 5.3.1.2). As inflow continues to increase, the gates are opened further to increase the outflow from the pools until the gates are entirely out of the water as shown in Figure 5-30c. After the flood crest passes and the flow recedes, the gates are then partially lowered into the water as required to restore the pools to normal operating levels as shown in Figure 5-30d. The instantaneous discharge, day of the flow cycle, and shape of the hydrograph are shown in each plot.

Using a similar format the changes in riverbed elevation in this reach of the Upper Mississippi River during the flood routing are shown in Figure 5-31. The difference between the solid and the dashed line indicates the changes in the riverbed elevation. It can be seen that the riverbed at a section does not continuously aggrade or degrade but fluctuates in response to the changing patterns of water and sediment discharge.

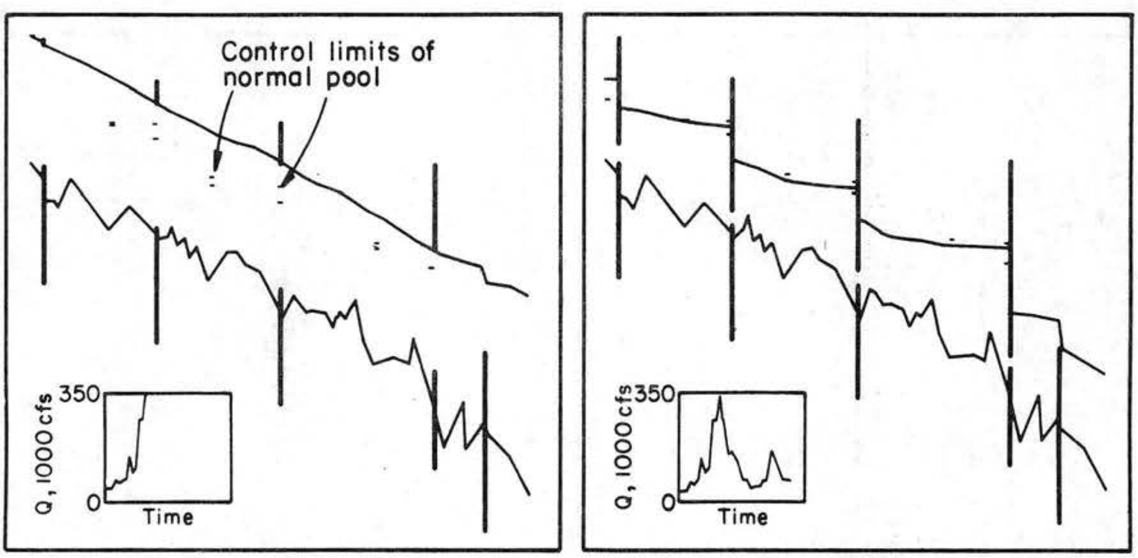
5.4.2 Predicting Long-Term Response

5.4.2.1 Effects of Continuing Existing Operating Procedures. The mathematical model of the study reach was operated to assess future geomorphic changes that would result if the existing scheme of operations



a. Normal pool stages with control gates lowered close to gate sills
Q=28,000 cfs at Dam 22, 24 days

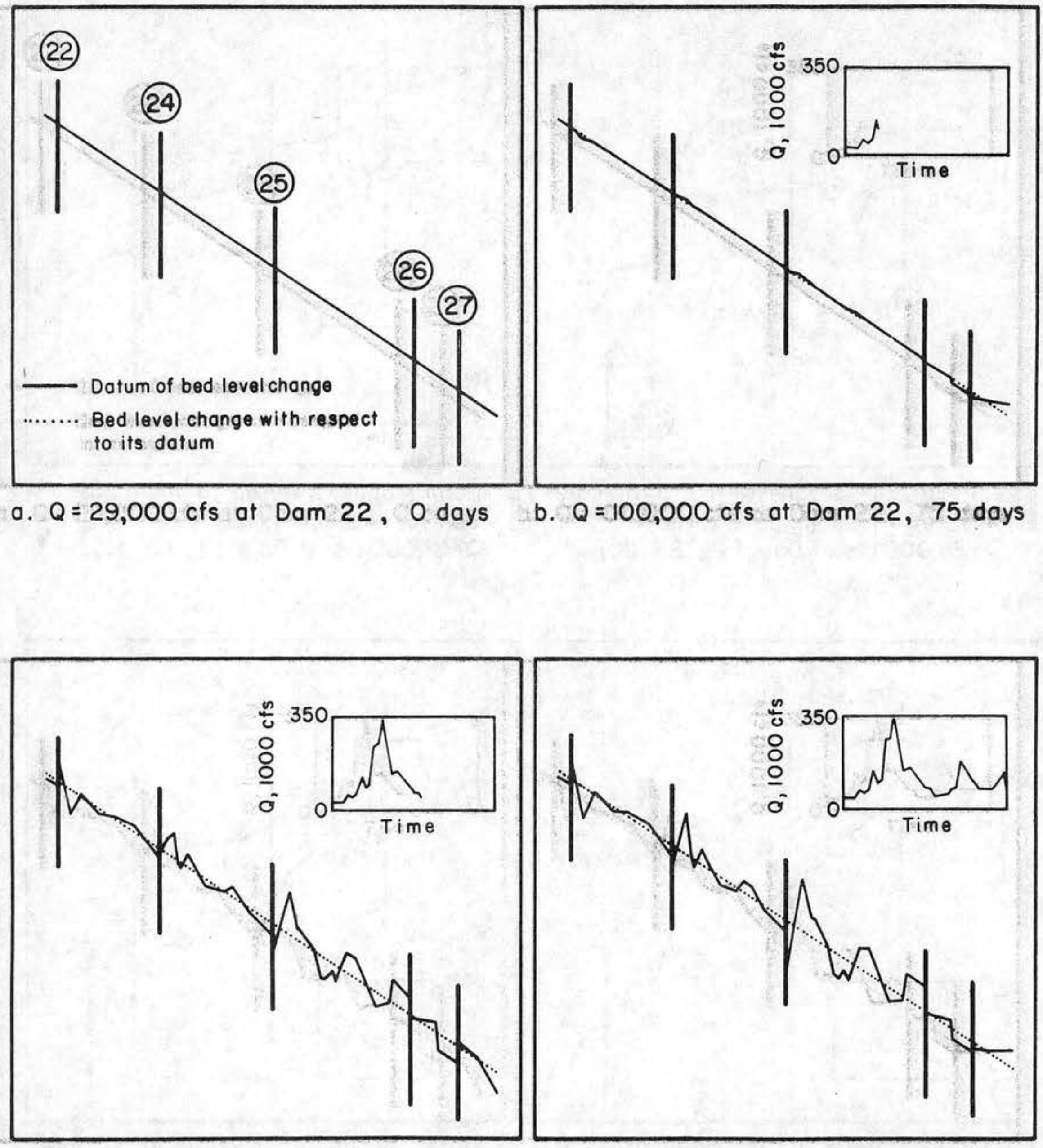
b. Pool stages lowered to maintain control station levels
Q=81,000 cfs at Dam 22, 63 days



c. Gates entirely out of water at flood crest
Q=339,000 cfs at Dam 22, 117 days

d. Gates partially lowered to restore pools after flood recedes
Q=54,000 cfs at Dam 22, 324 days

Figure 5-30 Water Surface Profiles in the Upper Mississippi River during a Flood Routing.



a. Q = 29,000 cfs at Dam 22, 0 days

b. Q = 100,000 cfs at Dam 22, 75 days

c. Q = 38,000 cfs at Dam 22, 207 days

d. Q = 51,000 cfs at Dam 22, 365 days

Figure 5-31 Bed Elevation Changes in the Upper Mississippi River during a Flood Routing.

to maintain the 9-foot channel were continued for 50 years. The hydrographs used in the model were synthesized from the 1932 to 1973 peak discharge and flow volume frequency curves. The sediment supply rates employed were those obtained in the calibration of the model.

The anticipated riverbed elevation changes in the Pool 24, 25, and 26 reach in the next 50 years are given in Table 5-6.

Table 5-6 Future Riverbed Elevation Changes in the Upper Mississippi River

Location	Riverbed Elevation Change after 1975,* ft				
	1985	1995	2005	2015	2025
Pool 26:					
Below Illinois River	1.1	0.4	0.2	0.6	0.6
Middle third	0.2	0.5	0.3	0.4	0.2
Next eighth	3.5	2.8	2.2	1.8	1.2
Upper eighth	-1.9	-3.7	-4.9	-6.0	-6.7
Pool 25:					
Lower quarter	-0.9	-0.7	-1.1	-1.1	-1.0
Lower middle quarter	0.1	-0.9	-0.3	-0.3	-0.2
Upper middle quarter	0.0	1.0	1.5	1.8	1.9
Next eighth	1.9	1.9	2.3	3.0	3.2
Upper eighth	-2.0	-2.7	-1.3	-2.6	-3.0
Pool 24:					
Lower quarter	0.1	1.1	0.7	0.7	0.3
Lower middle quarter	1.1	0.7	-0.5	-1.1	-1.2
Upper middle quarter	-0.2	-1.8	-2.8	-2.7	-2.7
Upper quarter	-1.5	-0.7	-1.1	-2.1	-2.4

*Positive changes signify aggradation and negative changes degradation of the riverbed.

In the upper half of Pool 24, the riverbed degrades until Year 2000 and remains essentially unchanged thereafter. The maximum bed degradation is about 2.5 feet below 1975 riverbed level. In the earlier years, the flow in the upper reach has the capability to carry more sediment than is released from Pool 22. Channel erosion results in enlarging the river cross section, which in turn reduces the flow velocity and the sand transport capability of the upper reach. After years of degradation, the channel conditions of the upper reach approach equilibrium. In the meantime, bed degradation begins slowly in the lower half of Pool 24 after 1995 and causes its aggraded bed to degrade.

In Pool 25, 3 feet of degradation occurs below Dam 24 in 50 years. Downstream of this degraded reach there is a reach of braided channel where the sediment transport capability is small. In this braided reach, 3 feet of bed aggradation is anticipated in the next 50 years.

It should be recalled that the riverbed generally fluctuates with time as sand waves moves downstream. Sediment may deposit on the riverbed during low or medium flow but the deposited sediment may be eroded away during the high flow or vice versa. In general however, the crossing areas accumulate sediment more readily than the other portions of a river reach. Therefore, at crossings the bed elevation fluctuates with a trend toward aggradation. The opposite occurs in the deep pools of the channel bends.

In the lower half of Pool 25, the riverbed degrades until Year 2000 because large amounts of sediment are being trapped in the upper reach. After Year 2000, the upper reach approaches an equilibrium state and passes a greater sediment load which, in turn, arrests the degrading of the lower reach.

In Pool 26, the riverbed immediately downstream of Lock and Dam 25 degrades for the entire 50 years. The lower end of the pool aggrades 3.5 feet within 10 years, then begins to degrade slowly.

Under existing operating procedures, the natural levees along the Upper Mississippi River bankline and islands (Section 3.5.2) continue to grow in the 50 years simulated. It is estimated that the natural levees along the Mississippi increase 2.5 feet in height in 50 years. Away from the natural levees, the deposition of sediments (mainly silts and clays) on the floodplain are not large. In the Pool 24 reach, for example, approximately 1 inch of silts and clays are deposited in 50 years.

From the results of the model study, it becomes apparent that the geomorphic change of the Pool 24, 25, and 26 reach is a slow process. Because of trap of sediment in the upstream locks and dams, in general, chute channels will remain open for a long period of time. However, backwater channels are being slowly filled with sediment. The mathematical model was not utilized to study evolution of an individual side channel. Special considerations and additional data can be added to the model to assess its geomorphic change (Dass, 1975).

5.4.2.2 Effects of Altering Existing Operation Procedures. To study the impacts of alternate operational procedures on the morphology of the Pool 24, 25, and 26 reach a number of alternatives were modeled, including: operating the navigation pools one foot above and one foot below normal levels, simulating zero sediment inflow to Pool 24, and simulating a maximum sediment inflow from Pool 22.

The geomorphic changes in the study reach caused by holding the pool levels 1 foot above the normal pool level for 50 years are not significantly different from operation at normal pool level. However, increasing the pool levels reduces the sediment transport capability of the river reach. The reach aggrades more and degrades less when the pools are held 1 foot higher than normal pool. The maximum difference in riverbed elevation is on the order of 1.0 feet in the degrading reach immediately below Lock and Dam 24. There is a 10-percent increase in floodplain deposits of silts and clays as a result of holding normal pool level 1 foot higher, but since these floodplain deposits are small, the increase is not significant. In addition, the natural levee heights are not increased significantly.

The response to holding pool levels 1 foot below the normal pool is not much different than for 50 years of operation at normal pool. The trends of the changes are similar, but decreasing the pool level increases transport capacity so that there is less aggradation and more degradation with the lower level. The maximum difference in riverbed elevation is an additional 0.7 foot of degradation in the reach immediately below Lock and Dam 24. Operation at lower than normal pool for 50 years reduces floodplain deposits but not significantly. Thus, changing the operating scheme for the locks and dams by raising or lowering the normal pool level by 1 foot has limited long-term effects on the morphology of the river and adjacent lands in the Pool 24, 25, and 26 reach.

Assuming it was possible to completely arrest the transport of sediment into Pool 24, the river in Pools 24, 25, and 26 would experience the maximum possible degradation. Zero sediment inflow to Pool 24 has been simulated assuming present-day operations of the dams over the next 50 years. Results are given in Table 5-7.

The maximum amount of degradation occurs in the upper reaches of Pool 24, and is calculated until the riverbed has degraded 6 feet. Greater degradation is possible, but in the absence of information on sand sizes below the bed surface, the calculations were terminated. Usually the bed material becomes coarser as the bed lowers, and degradation is arrested by the natural armoring effect of the coarser particles.

The effects of severe degradation in the upper reaches of Pool 24 do not reach the lower end of Pool 24 until Year 2015. Thus, completely

Table 5-7 Future Riverbed Elevation Changes in the Upper Mississippi River Assuming No Sand Transport into Pool 24

Location	Riverbed Elevation Change since 1975,* ft				
	1985	1995	2005	2015	2025
Pool 26:					
Below Illinois River	1.1	0.4	0.1	0.5	0.5
Middle third	0.2	0.5	0.2	0.4	0.1
Next eighth	3.5	2.9	2.2	1.8	1.2
Upper eighth	-1.9	-3.7	-5.0	-6.0	-6.7
Pool 25:					
Lower quarter	-0.8	-0.8	-1.1	-1.0	-1.1
Lower middle quarter	0.1	-0.8	-0.3	-0.3	-0.2
Upper middle quarter	0.0	1.0	1.6	1.8	2.1
Next eighth	1.9	1.9	2.2	2.9	3.2
Upper eighth	-2.1	-2.9	-1.6	-3.0	-4.0
Pool 24:					
Lower quarter	0.1	1.1	0.6	0.5	-0.1
Lower middle quarter	1.0	0.6	-1.1	-2.0	-3.0
Upper middle quarter	-0.3	-2.6	-5.1	-5.9	--
Upper quarter	-4.3	--	--	--	--

*Positive and negative changes signify aggradation and degradation respectively.

stopping the flow of sediment into the study reach for the next 50 years affects Pool 24, but has very little effect on riverbed elevations in Pools 25 and 26.

If the control gates of Lock and Dam 22 were raised out of water at all times or if Pool 22 were to become filled with sediment, the rate of sand transport into Pool 24 would be the maximum. In this case, the riverbed in Pool 24 aggrades 3 feet in the next 50 years as shown in Table 5-8. However, the effects of discharging the maximum sand load into Pool 24 do not go beyond Lock and Dam 24 for the next 50 years. The geomorphic changes in Pools 25 and 26 in the next 50 years are quite similar to that obtained with normal sand loads.

It is clear, then, that any change in the delivery of sediment and water from upstream of Lock and Dam 22 to the study reach does not significantly affect the morphology of the river and adjacent lands below Lock and Dam 24, at least in a period of 50 years.

Table 5-8 Future Riverbed Elevation Changes in the Upper Mississippi River Assuming Maximum Sand Transport into Pool 24

	Riverbed Elevation Change since 1975,* ft				
Location	1985	1995	2005	2015	2025
Pool 26:					
Below Illinois River	1.2	0.5	0.2	0.6	0.6
Middle third	0.2	0.6	0.3	0.4	0.1
Next eighth	3.6	3.0	2.2	1.9	1.3
Upper eighth	-1.9	-3.7	-5.0	-5.8	-6.7
Pool 25:					
Lower quarter	-0.9	-0.8	-1.0	-1.3	-1.4
Lower middle quarter	0.1	-0.8	-0.3	-0.3	-0.1
Upper middle quarter	0.0	1.1	1.6	1.9	2.3
Next eighth	1.9	2.0	2.3	3.2	3.5
Upper eighth	-1.9	-2.8	-1.2	-2.2	-3.0
Pool 24:					
Lower quarter	0.1	1.1	1.3	1.5	1.8
Lower middle quarter	1.1	1.2	1.2	1.4	2.2
Upper middle quarter	0.0	0.0	0.1	1.2	1.5
Upper quarter	1.7	2.5	2.7	2.9	2.5

*Positive and negative changes signify aggradation and degradation respectively.

5.4.3 Summary

Future geomorphic changes in a specific river reach can be assessed on the basis of information derived from the study of past geomorphic changes and with the aid of a mathematical model of the river reach in question. On the basis of a study of past geomorphic changes in Pools 24, 25, and 26 supported by a mathematical simulation of future river response, it is concluded that the river scene in this reach 50 years into the future will be essentially as it is today. The present-day manner of operation does not have any serious detrimental effects on the geomorphology or hydraulics of the river system in the reach. Future geomorphic changes that will occur in Pools 24, 25, and 26 in the Upper Mississippi River due to present and anticipated future developments include:

1. If the pools are operated in the present-day manner for the next 50 years and if the sediment load to the study reach remains essentially unchanged, the riverbed in Pool 24 will have degraded approximately 1.5 feet overall, Pool 25 will have degraded 3.0 feet immediately below Lock and Dam 24, aggraded approximately 3.0 feet immediately upstream of the pool control point and will remain unchanged in the lower portion of the pool. Pool 26 will have degraded between 6 and 7 feet immediately downstream of Lock and Dam 25.

2. Under the present-day manner of operation the natural levees along the riverbanks and on the islands of the Upper Mississippi will grow on the average 2.5 feet in height.

3. Under the existing mode of operation, approximately one inch of silts and clays will be deposited on the unprotected floodplains along the study reach of the Upper Mississippi River.

4. The geomorphic changes caused by operating the pools one foot above normal pool for 50 years are not significantly different from operation at normal pool level. Increasing the pool levels causes aggrading reaches to aggrade more and degrading reaches to degrade less.

5. Holding the pools one foot above normal for 50 years causes increased deposits on the natural levees and on the floodplains but these increases are not significant.

6. The effects of holding pool levels one foot below normal pool level for 50 years are not much different than for operation at normal pool level. Decreasing the pool level results in less aggradation and more degradation.

7. Holding the pools one foot lower than normal for 50 years results in smaller deposits on the floodplain and natural levees but again, this is not a significant factor.

8. If no sediment is supplied to Pool 24, the river would degrade severely in Pool 24 but there would be very little effect on riverbed elevations in Pools 25 and 26.

9. With the maximum sand transport rate passing Lock and Dam 22 into Pool 24, the riverbed in Pool 24 would aggrade 3 feet in the next 50 years. However, the effect would not go beyond Lock and Dam 24, at least for 50 years.

Chapter 6

RIVER RESPONSE TO CHANNEL STABILIZATION

6.1 Introduction

From a study of the chapters on river mechanics, river morphology, and river response it should be clear that both short- and long-term changes can be expected on river systems as a result of natural and man-made influences. The hydraulics of river training and common methods of river control are reviewed in this chapter, with particular emphasis on the impact of contraction dikes and revetment on the river environment. The related and environmentally critical problem of the evolution of man-made side channels is presented in the light of both field experience and laboratory hydraulic model studies. Finally, the integrated and interactive effects of contraction works on the river environment are illustrated by an examination of the hydraulic and geomorphic response of the Middle Mississippi River to development.

Numerous types of river control and bank stabilization devices have evolved through past experience. Concrete, brick, willow and asphalt mattresses, sacked concrete and sand, riprap, grouted slope protection, sheet piles, timber piles, steel jack and brush jetties, angled and sloped rock-filled, earth-filled, and timber dikes, automobile bodies, and concrete tetrahedrons have all been used in the practice of training rivers and stabilizing river banks. An extensive treatise on the subject of bank and shore protection was prepared by the California Division of Highways (1970). A large number of publications on river training and stabilization have been prepared by the Corps of Engineers and the U.S. Bureau of Reclamation. Many more publications on the subject exist in the open literature. It is not intended that an exhaustive coverage of the various types of river control structures and methods of design be made in this handbook. Rather, it is the purpose to review various methods of channel stabilization and examine the impact of those methods most commonly used on the Upper Mississippi Rive

6.2 The Hydraulics of River Training

6.2.1 The Objectives of Channel Stabilization

Lindner (1969) provides an excellent summary of the objectives and most common methods of regulating and stabilizing alluvial channels. Basically, there are two fundamental reasons for employing channel stabilization works: 1. protection, i.e., protection of properties from erosion and from floods, and 2. improvement of channels for navigation. For protection from erosion and floods, channel realignment, revetments, dikes, groins, levees, retards, and bankheads are employed. For navigation purposes similar measures and structures are used to provide proper alignment and to deepen the shallows. A supplementary purpose for confinement and deepening an alluvial river, important in arid regions, is the reduction of the water surface area available for evaporation.

Except for revetments, works for protection will generally be located at different points than works for navigation. Navigation works are intended primarily to deepen the channel over shoal areas which may or may not require bank protection from scour. Features of the design may also vary. For example, protective structures placed in concave banks of bendways are subject to more violent and direct attack than structures placed elsewhere. Accordingly they must be built stronger and, in the case of dikes, be more closely spaced or aligned to withstand the attack. It may be desirable for the heights of the structure to be different also. For navigation purposes the concave banks need only be stabilized to provide or retain a favorable alignment whereas for protective purposes the foreshore and the levee slope may require protection as well.

6.2.2 Channel Alignment

Channels requiring realignment because of instability, excessive curvature, or the need to protect contiguous properties usually are laid out on topographic maps or aerial photographs, taking into account surface and subsurface conditions. Where complete freedom of location exists, the channel should be planned as a series of reversing curves connected by short tangents. For stabilization purposes the radii of the bends should be selected from an examination of the river to ascertain the curvature that will not subject the banks to extreme attack and yet will be sufficient to hold the thalweg in a fixed location.

Though this curvature may be ideal from the channel and bank stabilization point of view, it may not produce a satisfactory channel for navigation. Short radius bends are accompanied by comparatively deep and narrow channels; long radius bends have wider and shallower channels in which the thalweg may swing away from the concave bank, resulting in an irregular pattern of bar formation. An optimum bend for navigation purposes is one that produces the width and depth desired, provided the curvature is not excessive in relation to the operational capabilities of the rivercraft involved.

The length of the tangents between bendways should also be determined from a study of the river to be improved and the characteristics of tows expected to utilize the channel. In planning the tangent lengths between reversing bends, cognizance must be taken of the fact that the principal current must cross the channel centerline in moving from the upstream bend to the concave side of the next downstream bend. An acute angle between this current and the bank of the downstream bend will require high maintenance or more costly works where the current impinges on the bank. The area of most serious attack will depend upon the angle. Generally this angle should be limited to a maximum of 15 degrees though, where this is impracticable, angles up to 25 degrees may be used (Lindner, 1969). The minimum tangent lengths that are likely to retain this angle below the limits mentioned and to adequately accommodate navigation should be selected. This is especially important since long tangents may result in additional attempts of the river to meander, eventually upsetting the planned alignment or causing undue expense to retain it. Such tangents often are accompanied by shallow stretches which are difficult and costly, if not impossible, to maintain satisfactorily for navigation.

The optimum channel width based solely on channel stabilization requirements is the maximum which will provide a cross section free from obstructing bar formations at normal low discharge. A channel which is too narrow will be subject to excessive bed scour and possibly undermining of control works; one which is wide enough to permit random shifting of flow lines will result in the formation of middle bars that are unstable as to location and configuration. Such a channel will be inefficient, and points of attack will be unpredictable and variable.

Where a channel is completely realigned to control curvature, width, and depth, the new channel position will usually have to be maintained by revetment or dikes to prevent erosion in the new bends, and by longitudinal or lateral dikes to close off old channels, concentrate the flow, and resist the tendency of the river to return to its old alignment. Complete relocation of long stretches cannot be successfully accomplished and maintained without extensive regulation and control works.

In realigning a river channel, a sinuous channel may be made less sinuous, or a straight channel may be modified to include one or more curves. The radii of curvature, the number of bends, the limits of rechannelization, hence the length or slope of the channel, and the cross-sectional area are decisions which have to be made by the designer. Different rivers have different characteristics and different geomorphic history with regard to channel migration, discharge, stage, geometry and sediment transport. As indicated in the previous chapters, it is important for the designer to understand and appreciate river hydraulics and geomorphology when making decisions concerning channel improvement. It is difficult to state generalized criteria for channel improvement applicable to any river. Knowledge about river systems has not yet advanced to such a level as to make this possible. Nevertheless, it is possible to provide some principles and guidelines for channel improvement.

As the general rule, the radii of bends should be made about equal to the mean radii of bends, r_c, in extended reaches of the river. As noted above, the angle α shown in Figure 6-1 between a line drawn tangent to the inside of two successive bends and the bank line in the crossing should be approximately 20 degrees. This provides a sufficient crossing length for the thalweg to swing from one side of the channel to the other. Generally, it is necessary to stabilize the outside banks

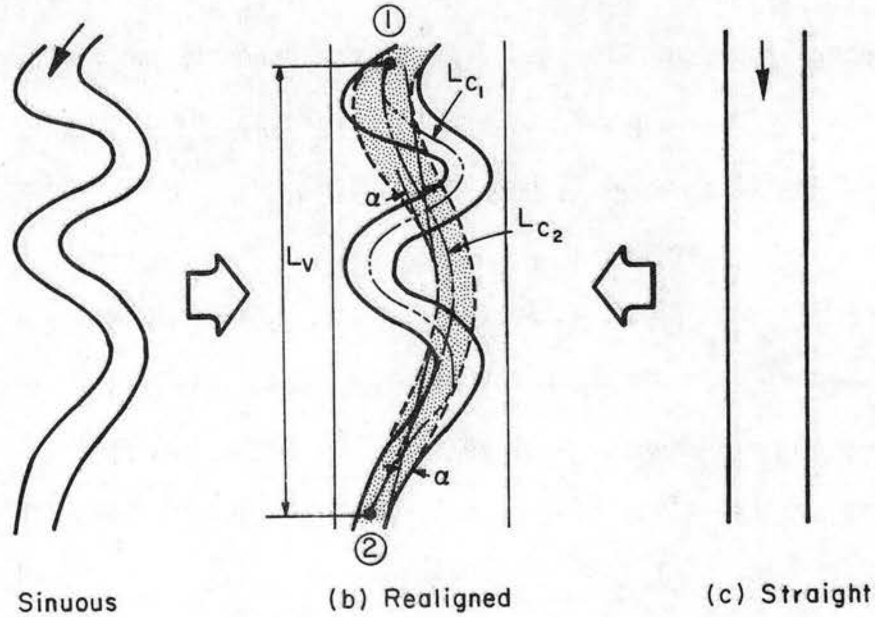


Figure 6-1 Realignment of River Channels.

of the curves in order to hold the new alignment. Depending upon crossing length, some amount of maintenance dredging may be necessary to remove sandbars after large floods so that the channel does not develop new meander patterns in the crossings during normal flows.

As an illustrative example consider the realignment of the very sinuous river of Figure 6-1a to the less sinuous river of Figure 6-1b. The sinuosity and channel bed slope are related in the following manner. The bed elevations at the ends of the reach being rechannelized, designated 1 and 2, in Figure 6-1b are established by existing conditions. Hence, the total drop in bed elevation for the new channel (subscript 2) and the old channel (subscript 1) are the same.

$$\Delta z_1 = \Delta z_2 = \Delta z \tag{6.1}$$

The length of channel measured along the thalweg is labeled L_c. Thus, the mean slope of the channel bed before rechannelization is

$$S_1 = \frac{\Delta z}{L_{c_1}} \tag{6.2}$$

and after rechannelization is

$$S_2 = \frac{\Delta z}{L_{c_2}} \tag{6.3}$$

Sinuosity (s) is defined by the ratio of the length of channel to length of the valley, or

$$s = \frac{L_c}{L_v} \geq 1 \tag{6.4}$$

Clearly,

$$s_1 = \frac{L_{c_1}}{L_v} \tag{6.5}$$

$$s_2 = \frac{L_{c_2}}{L_v} \tag{6.6}$$

Thus,

$$s_1 S_1 = \frac{L_{c_1}}{L_v} \cdot \frac{\Delta z}{L_{c_1}} = \frac{L_{c_2}}{L_v} \cdot \frac{\Delta z}{L_{c_2}} = s_2 S_2 \tag{6.7}$$

The new channel slope and channel sinuosity are inversely related. If $s_2 < s_1$ then $S_2 > S_1$. The new channel alignment, hence s_2, can be chosen by the designer with due consideration given to the radii of curvature, deflection angles and tangent lengths between reversing curves. As indicated before, consideration should also be given to prevailing average conditions in the extended reach. The new slope S_2 can be calculated from Equation (6.7), and the relationship (from Equation 3.2)

$$S_2Q^{1/4} \leq 0.0017 \tag{6.8}$$

should be satisfied. If S_1 is of such magnitude that Equation (6.8) cannot be satisfied with still larger S_2, the possibility of the river changing to a braided channel because of steeper slope should be carefully evaluated. With steeper slope, there could be increase in sediment transport which could cause degradation and the effect would be extended both upstream and downstream of the rechannelized reach. The meander patterns could change. Considerable bank protection might be necessary to contain lateral migration which is characteristic of a braided channel, and if the slope is too steep, head cuts could develop which migrate upstream with attendant effects on the plan geometry of the channel. Even when changes in slope are not very large, a short-ter adjustment of the average river slope occurs, consistent with the sediment transport rate, flow velocities and roughnesses, beyond the upstream and downstream limits of channel improvement. For small changes in slope, the proportionality (Equation 3.9), $QS \sim Q_s D_{50}$ tends toward equilibrium by slight increases in bed material size D_{50} and adjustment in the sediment transport rate Q_s.

A small increase in the new channel width could be considered so as to maintain the same stream power in the old and new channels. That is,

$$(\tau_o V)_1 = (\tau_o V)_2. \tag{6.9}$$

With substitution of $\tau_o = \gamma RS$, $V = Q/A$ and $R = A/P \simeq A/W$, Equation (6.9) leads to

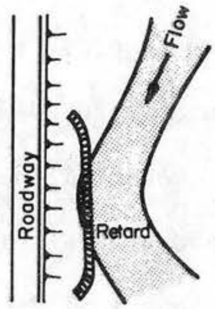
$$W_2 = \frac{S_1}{S_2} W_1 \tag{6.10}$$

The increase in width should be limited to about 10 to 15 percent. Wider channels would be ineffective. Deposition would occur along one bank and the effort of extra excavation would be wasted. Furthermore, bar formation would be encouraged, with resultant tendencies for changes in the meander pattern leading to greater maintenance costs of bank stabilization and removal of the bars to hold the desired river alignment. The depth of flow in the channel is dependent on discharge, effective channel width, sediment transport rate (because it affects bed form and channel roughness) and channel slope.

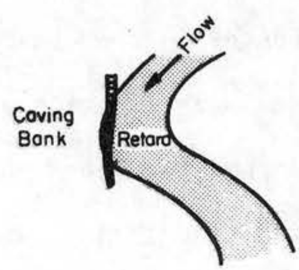
The foregoing discussion pertains to alluvial channels with fine-sized bed materials (sands and silts). For streams with gravel and cobble beds, the concern is to provide adequate channel cross-sectional dimensions to convey flood flows. If the realigned channels are made too steep, there is an increased stream power with a consequent increase in transport rate of the bed material. The deposition of material in the downstream reaches tends to form gravel bars and encourages changes in the plan form of the channel. Short-term changes in channel slope can be expected until equilibrium is reestablished over extended reaches both upstream and downstream of the rechannelized reach. Bank stabilization may be necessary to prevent lateral migration, and periodic removal of gravel bars may also be necessary.

6.2.3 River Training Structures

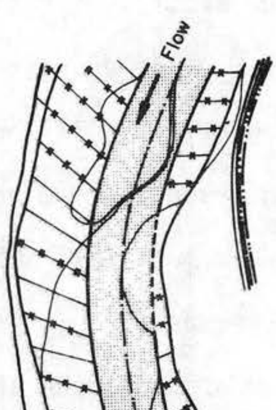
Various devices and structures have been developed to control river flow along a preselected path and to stabilize the banks. Most have been developed through a trial and error process, aided in some instance by hydraulic model studies. Rock riprap is probably the most widely used material to stabilize river banks and protect the side slopes of embankments. Dikes, retards, and jetties are devices used to guide the river flow and to protect the banks. Their use is illustrated in Figure 6-2. In all instances, the intent of the devices shown in



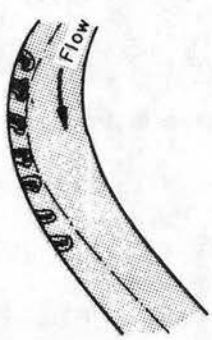
(a) Retards to protect highway embankment



(b) Retard to prevent further bank caving



(c) Jetties to train the flow and protect the bank



(d) Dikes to train the flow and protect the bank

Figure 6-2 Retards, Jetties, and Dikes to Protect Embankments and Train Channel Flow.

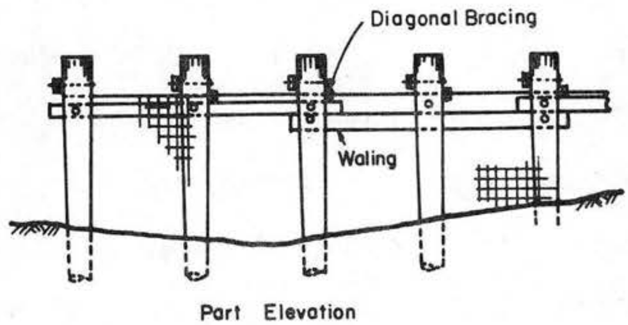
Figure 6-2 is to cause resistance or obstruction to flow along the channel bank, thereby creating lower velocities to prevent bank erosion, restrict the thalweg of the stream to the center portions of the channel, and if possible cause deposition of sediment along the bank where erosion previously occurred.

The selection of the type of structures to be used in river training depends on many factors. Most important, perhaps, is the success experienced with a given type of structure. In judging the success or lack of success in past use, the characteristics of the streams on which experience data is available should be compared with those of the stream for which the structure is being considered. For instance, what is the difference in the sediment and the sediment transport; what are the slopes, the velocities, the discharges, the detritus carried (other than sediment), and the angles of attack? Will these have an influence on the effectiveness of the structure? If revetments are to be used, what types can be expected to be successful and which is the most economical considering cost of materials, difficulty of placement, and expected maintenance? Other methods for accomplishing the development objectives should also be considered. For example, dredging may be less costly on an annual basis than dikes or revetments, or a combination of dredging, dikes and revetment may be most economical. In the case of dredging, consideration must be given not only to cost, but to the feasibility of timely accomplishment of the work and problems of dredged material disposal. The availability and accessibility of local materials affect the practicability and the cost of construction, and may also be determining factors in the economic analysis.

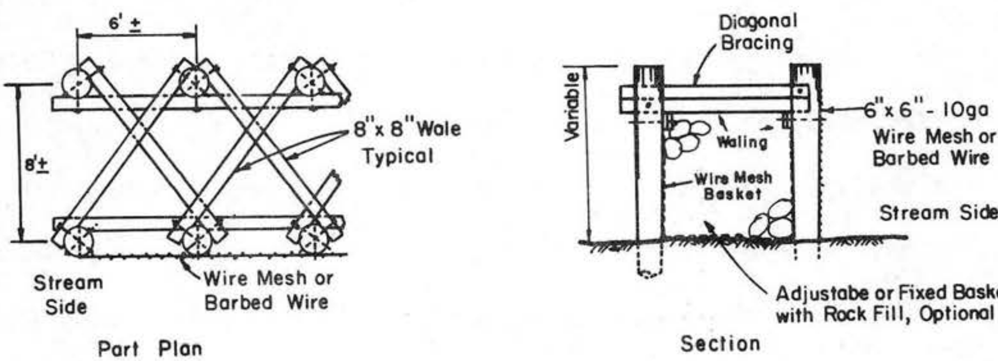
6.2.3.1 Dikes. In general dikes extend outward from the bank into the channel at right angles or angled thereto, depending upon the circumstances and particular success achieved in past applications. Along straight reaches, dikes should be perpendicular to the bank. Along sharp curves the dikes should be angled slightly downstream so as to deflect the flow toward the center of the channel. Some dikes are terminated with extensions parallel to the flow, forming L or T shapes, and are correspondingly referred as L or T head dikes.

There are two principal types of dikes, permeable and impermeable. Permeable dikes are those which permit flow through the dikes, but at reduced velocities, thereby preventing further erosion of the banks and causing deposition of suspended sediment from the flow. Impermeable dikes are designed to protect banks and confine the flow by deflecting the flow away from the bank. Deposition can occur in an impermeable dike field and increase the protective capability of these structures.

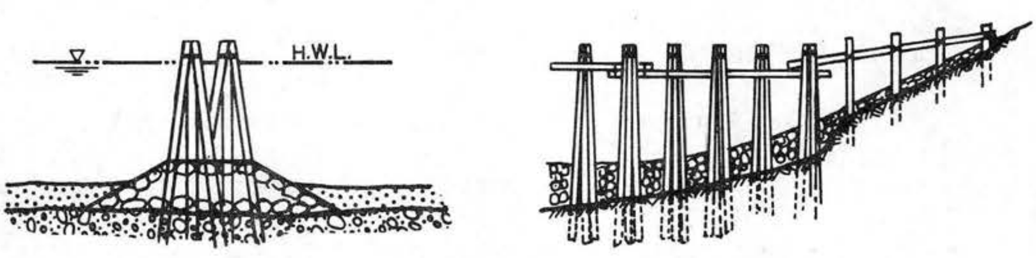
Timber pile dikes - Timber pile dikes are permeable dikes that usually consist of closely-spaced single, double, or multiple rows. There are a number of variations to this scheme. For example, wire fence may be used in conjunction with pile dikes to collect debris and thereby cause effective reduction of velocity. Double rows of timber piles can be placed together to form timber cribs, and rocks may be used to fill the space between the piles. Timber pile dikes are vulnerable to failure through scour. To prevent scour the piles can be driven to greater depth or the base of the piles can be protected from scour with a dumped rock blanket, forming a combination permeable and impermeable dike. The various forms of timber pile dikes are illustrated in Figure 6-3.



(a) Single row timber pile with wire fence



(b) Double row timber piles with rocks and wire fence



(c) Pile clusters

Figure 6-3 Timber Pile Dikes.

The arrangement of timber piles depends upon the velocity of flow, quantity of suspended sediment transport, and depth and width of river. If the velocity of flow is large, timber pile dikes are not likely to be very effective. Stabilization of the bank by other methods should be considered. On the other hand, in moderate flow velocities with high concentrations of suspended sediments, these dikes can be quite effective. Deposition of suspended sediments in the pile dike field is a necessary consequence of reduced velocities. If there is not sufficient concentration of suspended sediment in the flow, or the velocities in the dike fields are too large for deposition, the permeable timber pile dikes will be only partially effective in training the river and protecting the bends.

The length of each dike depends on channel width, position relative to other dikes, flow depth and available pile lengths. Generally, pile dikes are not used in large rivers where depths are great, although timber pile dikes have been used in the Columbia River. On the other hand, banks of wide shallow rivers can be protected with pile dikes. The spacing between dikes varies from 3 to 20 times the length of the upstream dike, with closer spacing favored for best results.

Stone-fill dikes - Stone-fill dikes are classed as impermeable dikes and, as noted, do not depend primarily on deposition of sediment between dikes to provide protection. As their principal function is to deflect the flow away from the bank, impermeable dikes must be long enough to accomplish this purpose. The dikes may be angled downstream, angled upstream, or constructed normal to the bank. Variations such as a sloping dike, with declining top elevation away from the bank, L or T head dikes, and curved dikes have been used. Stone-fill dikes are illustrated in Figure 6-4.

The spacing between dikes may vary from three or four dike lengths to 10 or 12 dike lengths depending upon velocity and depth. Short dikes

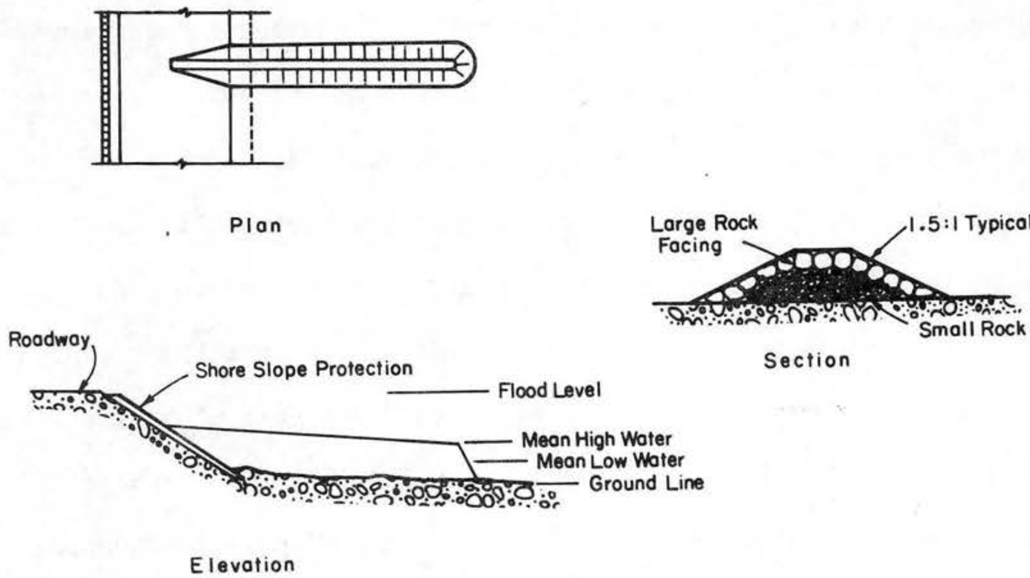


Figure 6-4 Typical Stone Fill Dike

with long spacing are generally not useful for bank protection unless jacks or riprap are used to protect the bank between them.

The ends of the dikes are subjected to local scour and appropriate allowance should be made for loss of dike material into the scour hole. The size of rock to be used for the dike depends on availability of material. Large rocks are generally used to cover the surface, while the internal section may consist of smaller rocks or earth-fill. Side slopes of 1.5:1 and 2:1 are common.

A summary of early developments on the Mississippi River above Cairo (Section 5.3.1.1) indicates that both permeable and impermeable dikes have been used for bank protection and development of the navigation channel. A review of dike construction experience on the Middle Mississippi River (St. Louis to Cairo) by Hartke (1966) provides valuable insights into the performance of the various types of contraction works.

As Hartke notes, in the building of constraction works, the cost of the structure relative to its effectiveness, and the availability of materials have always dictated the type of construction. Prior to 1900 engineers on the Middle Mississippi used stone piled on brush for dike construction becuase both of these materials were inexpensive, readily available, and formed an effective, somewhat impermeable dike. Silt ratios were high and the dikes silted in before the brush deteriorated.

Brush and stone dikes performed well and became the accepted method of contraction for many years. Brush was available along the entire river, but stone was not available in the delta area south of Cape Girardeau. Mobilization costs for quarry operation coupled with the problems of transporting large quantities of stone restricted the use of brush and stone dikes. Another limiting factor was the pyramiding quantities of materials, and accompanying costs, in constructing dikes in deep water. Problems were also encountered with this type of construction in fast currents. When velocities were high, brush was carried away during construction. Unless the dike slowed the water sufficiently to allow the deposition of silts on the brush, the dike soon deteriorated and was swept away.

Timber pile dikes were also excellent structures in their era. They were the dikes that accomplished the major portion of constriction of the Middle Mississippi River into a single navigation channel. Initially, they were single rows of timbers laced with brush. Need and experience gradually developed these simple structures into pile-clump dikes. They consisted of clumps formed by three piles, driven into the riverbed in rows at approximately 15-foot centers. The piles in the clumps were cabled to each other and to the horizontal piles (stringers) which bound the whole structure together. Many timber dikes are still in use on the Middle Mississippi, but as their effectiveness decreases, they are being converted by filling them with stone. Timber pile dikes are still being constructed in other areas where foundation problems exist, where water depths are large, and where there is not an economical supply of stone.

The silt load at St. Louis has decreased from 920,000 tons daily in the period 1917-1932 to 304,000 tons daily in 1962. This has reduced timber dike effectiveness. Unless timber dikes are covered, they deteriorate within 5 years and are virtually ineffective in 15 years. The maintenance costs after the initial 5 years also spiral upward. Timber dikes, unless covered with silt, are not as effective in constricting the river as impermeable dikes.

Timber dikes are highly susceptible to structural failure due to the momentum of drift and ice floes. The break up of ice jams in 1962 and 1963 destroyed almost every portion of the timber dikes standing in the water between Cape Girardeau, Missouri and Cairo, Illinois. Extensive damage was also inflicted on the timber dikes between Grand Tower, Illinois and Cape Girardeau. Weakened timber dikes are most susceptible and are literally "pushovers" in heavy ice floes.

Towboats also take their toll of timber dikes. During low stages, when control of boats is lost they are pulled into the piling by the draw of water passing through the dikes. At higher stages, towboats often take shortcuts and run over submerged dikes. When steel towboats and barges collide with timber dikes, they break piling like matchsticks.

Solid stone dikes have been constructed at various locations on the Middle Mississippi River since the 1930's. Due to the high cost of transporting and placing the stone, their use was limited initially to short, low dikes of small cross-sectional area. In early 1961, however, an existing deteriorated timber dike was filled with "shovel run" stone for a length of approximately 3000 feet. The stone fill extended across a low alluvial plain, braided by numerous streams of water, all out of the main channel. This area was on the convex sid of a bend and extended for over two miles. Velocities were high. Numerous towheads were formed and eroded away as new and additional secondary channels developed. Prior to this, six timber dikes, including the existing one, had been built across this reach, but were never sufficiently effective to stabilize the area. By stoning this old timber dike, an impermeable, indestructible barrier across the area was created. Some water carried around the riverward end of the converted stone dike, but it firmly blocked off the secondary channels. These channels became filled with drift and silt. The stone dike also stoppe the development of new channels, and the general area experienced deposi tion of silts when the bank was overtopped. As the ground surface was elevated from the deposition of silts, willow growth developed which further accelerated silt deposition. The area upstream of the dike is now completely consolidated and covered with timber. It has built up t the same elevation as the dike. The water, instead of flowing through t area, now flows through the main channel where it improves navigation. The willows will continue to accept fill and in a few years the ground elevation will be raised sufficiently so that it can be used for agricultural purposes.

Stone dikes work excellently as baffles when built from the shore to the navigation channel. This reduces the width of the river at the dike. The river, as it squeezes through the narrower opening, attempts to retain its normal cross-sectional area. When the far bank is relatively stable or protected by revetment, the water scours out the bed of the river and produces a navigable channel. When the river has deepened and regained its cross-sectional area the scour will stop, but velocities will remain adequate to keep the bed load moving. Because stone dikes are virtually impervious, they work instantly, actually crowding the water over and producing scour as they are being built.

On the Middle Mississippi both "high" and "low" stone dikes have been built. The high stone dikes are designed to contract flows during medium stages. The dikes are built to a stage equivalent to about two-thirds bank-full, and are usually out of the water and ineffective during low stages. They function best where the river is wide at medium stages. They utilize the higher velocities during the medium stages to scour out a navigation channel which will last through the low-water season.

The low stone dikes are designed to work during low stages and have no effect at medium stages. They are usually built in concave bends where the water depths are in excess of 40 feet. Their purpose is to create additional width instead of the excess depth. They are built to an elevation of 12 feet below the low water of record, and as stages lower they become increasingly effective. The lowering stages confine the river into a smaller cross-sectional area. Thus, as stages continue to lower, the dike takes up a larger and larger percentage of the cross-sectional area of the river. This causes increasing amounts of water to be deflected against the bar opposite the dike, scouring it away and producing greater channel width.

High and low stone dikes have also been combined very effectively for correcting short-radius bends. The resulting sloping dikes are high at the bank and low at the river end. In short-radius bends, the deep, fast water is directly against the concave bank, constantly attacking it. This causes the lower portion of the bank to erode and the top portion to cave into the river. The high-low dikes placed along the outside bank of the bed deflect the current away from the original bend, creating a longer radius bend which eliminates the scouring action at the concave bank. The high portion of the dike causes the water to drop its silt adjacent to the bank, thereby stabilizing it. The low portion of the dike produces scour on the convex side of the bend, moving the channel into a good navigation alignment.

6.2.3.2 Jetties. The purpose of a jetty field is to add roughness to a channel or overbank area to train the main stream along a selected path. The added roughness along the bank reduces the velocity and protects the bank from erosion. Jetty fields are usually made up of steel jacks tied together with cables.

Steel jacks are basic triangular frames tied together to form a stable unit. The resulting framework is called a tetrahedron. The tetrahedrons are cabled together with the ends of the cables anchored to the bank. Wire fencing may be placed along the row of tetrahedrons. Various forms of steel jacks may be assembled. Two types are shown in Figure 6-5a. Jacks must be tied together with cable and must have tiebacks to deadmen set in the bank. Tiebacks should be spaced every 100 feet and space between jacks should not be greater than their width.

Both lateral and longitudinal rows of jacks are used to make up the jetty field as shown in Figure 6-5b. The lateral rows are usually

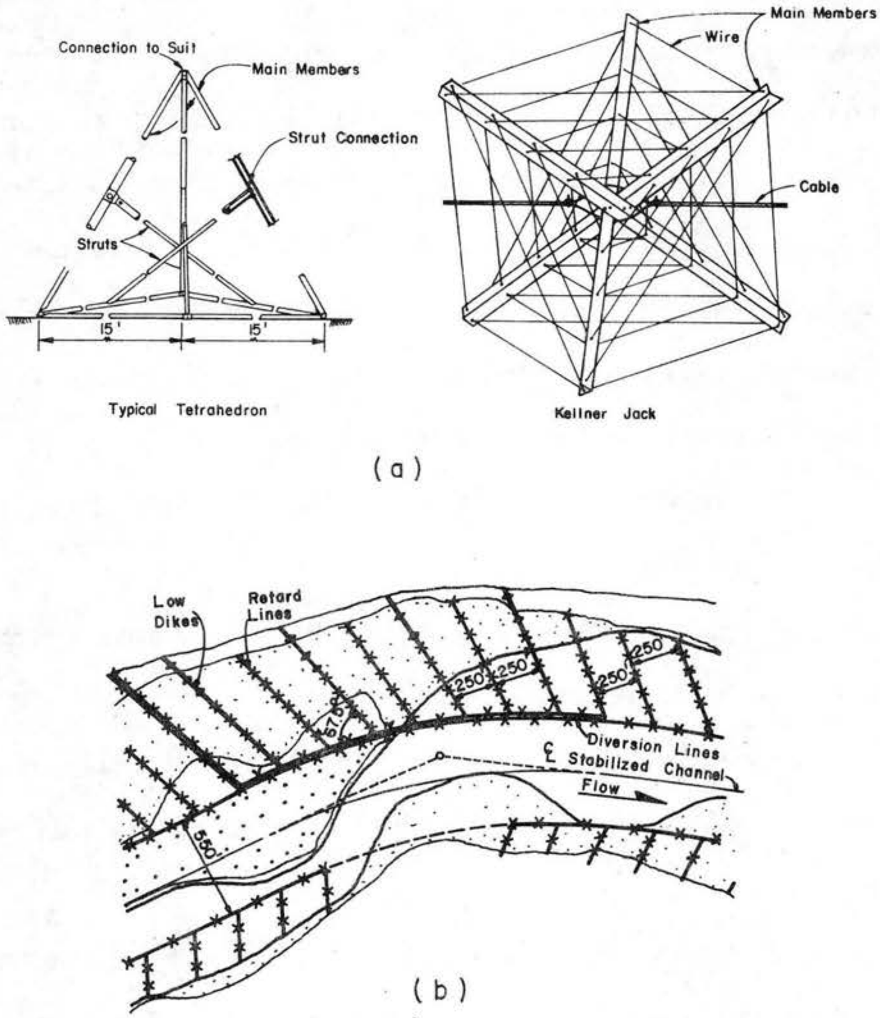


Figure 6-5 Steel Jacks and Typical Jetty-Field Layout.

angled about 45 to 70 degrees downstream from the bank. The spacing varies, depending upon the debris and sediment content in the stream, and may be 50 to 200 feet apart. Jetty fields are effective only if there is a significant amount of debris carried by the stream to collect on the fence, and the suspended sediment concentration must be large so that there will be deposition in the jetty field.

When jetty fields are used to stabilize meandering rivers, it may be necessary to use jetty fields on both sides of the river channel. Otherwise at flood stage the river may develop a chute channel across the point bar. A typical layout is shown in Figure 6-5.

6.2.3.3 Bank Protection. The term "bank protection" implies that the bankline has or is about to fail. In order to design bank protection properly, we must know how the bank fails. There are four principal ways in which a bank fails. They are:

1. The erosion of soil particles on the bank either by the river currents or by waves.

2. Sloughing banks caused by excessive internal hydrostatic pressure in the bankline materials.

3. Slip-circle failures caused by the undermining of the toe.

4. Liquefaction and subsequent movement of the soil mass (called a flow slide).

Bank protection revetments are generally, though not exclusively, located on the concave side of bends where bank recession as a result of the river currents is most active. They may be used elsewhere for wave wash prevention and protection of facilities such as railroad fills. According to the experiments of Friedkin (1945), bank protection works assist in deepening navigation channels in the bendways by resisting erosion of the banks and inducing erosion of the bed. Depending on the alignment and the upstream sediment supply, the depths over the crossing bars may be increased also. The action of deepening the bendway channel has long been observed in the field where undercutting of revetments has caused failure. Because of their high costs, revetments are not constructed for the sole purpose of channel deepening, except under special situations.

Because of conditions affecting construction and differences in the duration and intensity of attack, the segment of revetment placed above the approximate elevation of mean low water is frequently of different design than the segment placed below that elevation. The former is termed upper bank protection and the latter, subaqueous protection. Both are required to prevent bank recession. The upper bank protection may be extended to prevent attack by waves and currents on levees and other above-bank structures. For some types of bank protection, upper bank and subaqueous pavement are essentially the same and are placed in a single operation.

It is generally advantageous to grade the bankline upon which either upper bank or subaqueous protective structures are to be placed, although some protective features have been installed without grading with varying degrees of success. Even with those materials that performed well without grading, it was usually felt that better success could have been attained had the bank been graded. Generally, the banks should be graded to slopes that will be stable while the river is falling even when saturated. An exception would be when the banks are free draining or the material of which they are composed and the duration of the higher stages are such as to prevent saturation. Generally slopes of 1 vertical on 3 horizontal or flatter have been used, although in a few instances the slope has been graded as steeply as 1 on 2. The slopes to which the banks are to be graded will depend upon local conditions. They should be selected after a study of both the characteristics of the stream and the conditions of the soils upon which the structures are to be placed.

Numerous materials have been used for upper bank and subaqueous bank protection. Among these are hand-placed and dumped or cast quarrystone, gravel, cobbles, portland cement concrete, asphaltic concrete, branches and trees (especially willow), piling, lumber, metal, and various combinations of these materials. Several of the more common bank protection and revetment methods are discussed below.

Dumped or cast stone riprap, either graded or quarry-run, is now commonly used for upper bank paving. A desirable feature is its ability to settle and fill minor holes and otherwise adjust itself to irregularities that may develop in the subgrade. The cast stone approa is preferred since it avoids segregation of the riprap material that often accompanies the dumping process. If there is a deficiency in the smaller sizes, a gravel blanket or other filter material will be needed to prevent the loss of bank material between the stones. Quarry-run stone often contains a sufficient amount of the smaller sizes so a bedding blanket is not needed. When a bedding blanket is used, the particles should be angular rather than rounded or at least sufficiently off-round to prevent the pavement from slipping down the slope. The thickness of the pavement and the size of the stone to be used will depend on the severity of the attack.

On the Lower Mississippi graded stone ranging in size from 6 to 125 pounds and 10 inches in diameter is used on a 4-inch gravel blanket. In some instances on other streams larger rock has been used, especially where the upper and lower banks have been paved as a unit. On the Missouri River in Iowa and Nebraska the specifications provided that the thickness of the quarry-run, cast stone be 10 inches at the top of the slope increasing uniformly to 15 inches where it joins the trench fill, and the maximum size of stone be 250 pounds. No blanket underneat the stone is used (Lindner, 1969).

A unique method of placement of riprap was used along the Lower Colorado River near Needles, California. Here the river was to be channelized and stabilized on a planned alignment. Rock was windrowed along the desired bankline, the relocated channel was dredged, and the river allowed to cut its banks until the windrow was reached. The rock then dropped down the eroded bank, stopped the cutting action, and retained the bankline along the planned alignment. One and one-half to two cubic yards of stone per linear foot were windrowed with the view of obtaining a riprap thickness of two feet normal to the slope. The larger quantities were used along the concave banklines. The banks of the Lower Colorado are composed of sand and silt. This is an important factor, as the method was successful only where the banks were of noncohesive, alluvial materials. The size of the rock, which was quarry-run, was also important. It varied from 1/10 cubic feet to 1/4 cubic yard. The banks averaged 8 to 12 feet above the bottom of the dredged channel. The slope of the rectified channel is about 1-1/2 feet per mile and the water velocities average from 3 to 6 feet per second.

Rock-fill trenches are structures used to protect banks from caving caused by erosion at the toe. A trench is excavated along the toe of the bank and filled with rocks as shown in Figure 6-6. As the stream bed adjacent to the toe is eroded, the toe trench is undermined and the rock fill slides downward to pave the bank. The size of trench to hold the rock fill depends on expected depths of scour. It is advantageous to grade the banks before paving the slope with riprap and placing rock in the toe trench. The slope should be at such an angle that the saturated bank is stable while the river stage is falling.

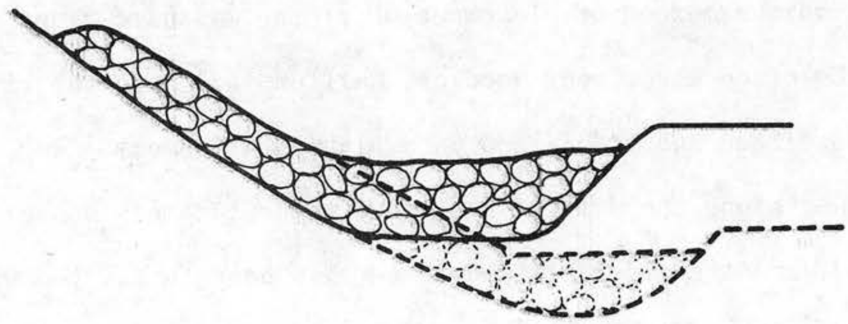


Figure 6-6 Rock-Fill Trench.

The rock-fill trench need not be at the toe of the bank. An alternative method is to excavate a trench above the water line along the top of the river bank and fill with rocks. Then as the bank erodes toward the trench, the rocks in the trench slide down and pave the bank. This method is applicable in areas of rapidly eroding banks of medium to large size rivers.

Rock-and-Wire Riprap - When adequate riprap sizes are not available rocks of cobble sizes may be placed in wire mesh mats made of galvanized fencing and placed along the bank forming a mattress. The individual wire units are called baskets if the thickness is greater than 12 inches The term mattress implies a thickness no greater than 12 inches. Toe protection is offered by extending the mattresses into the channel bed as shown in Figure 6-7. As the bed along the toe is scoured, the mattress drops into the scour hole. Special wire baskets of manageable sizes are manufactured and sold throughout the United States. It should be noted that when rock-and-wire mattresses are used in streams transporting cobble and rocks, the wires of the basket can be cut rather rapidly, which will destroy the intended protection along the base of the bank. Rusting of the wire mesh may also be a problem.

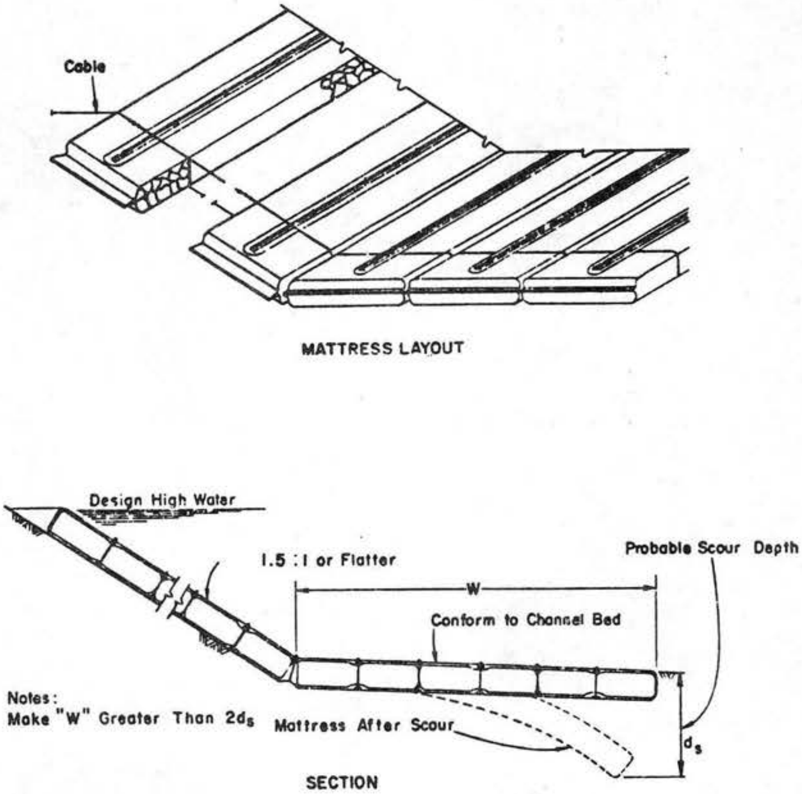


Figure 6-7 Rock and Wire Mattress.

Mats can be made up in large sizes in the field. The mats are flexible and can conform to scour holes which threaten the stability of the banks. The mats should be linked together to prevent separation as subsidence takes place.

Articulated concrete mattress - Small precast concrete blocks held together by steel rods or cables can be used to form a flexible mat as shown in Figure 6-8.

The sizes of blocks may vary to suit the contour of the bank. It is particularly difficult to make a continuous mattress of uniform sized blocks to fit sharp curves. The open spacing between blocks permits removal of bank material unless a filter blanket of gravel or plastic filter cloth is placed underneath. For embankments that are

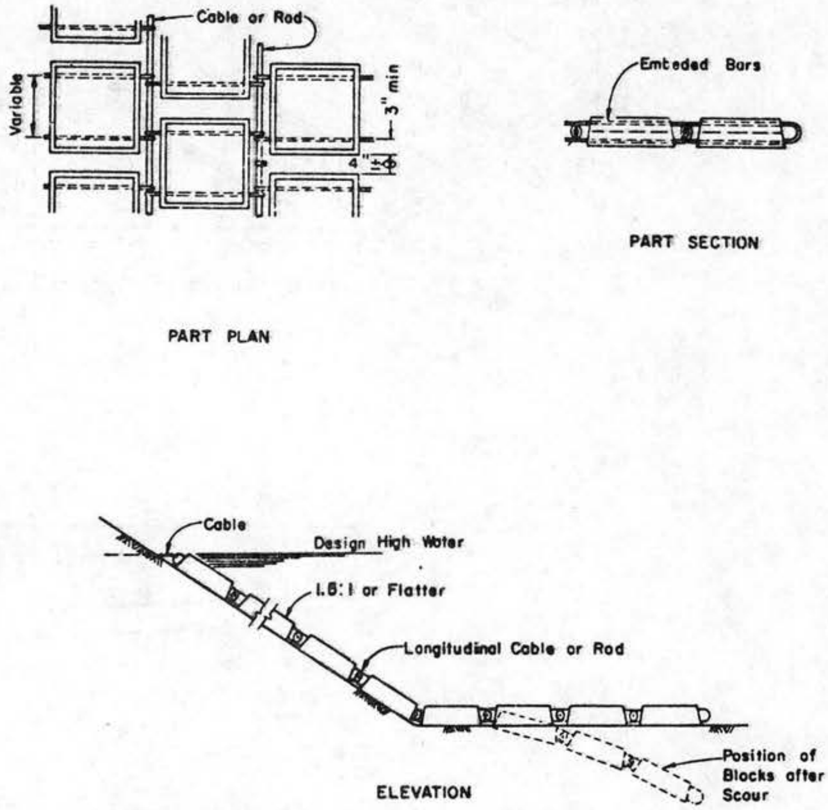


Figure 6-8 Articulated Concrete Mattress.

subjected only to occasional flood flows, the spaces between blocks may be filled with earth, and vegetation can be established.

The use of articulated concrete mattresses has been limited primarily to the Mississippi River. This is due to the large cost of the plant required for the placement of the mattress beneath the water surface. Thus, it is economically feasible to use articulated concrete mattresses only on rivers which require extensive bank protection. An expensive installation plant is not required, however, for placement of articulated concrete mattresses above the water surface. Paving the upper bank with articulated concrete mattresses has been used occasionally in the United States and Europe.

Other types of mattresses - Woven willow, brush, woven lumber, asphalt, and soil cement mattresses are other types that can be utilized. Concrete paving slabs are occasionally used. While paving slabs may be satisfactory along high water lines, they are not satisfactory for use below normal water levels because with even minor scour of the bank or toe, the rigid paving cannot conform to the scour hole and soon becomes undermined and the paving may break up. If the paving is extended well below the stream bed, or if the entire cross section is lined with concrete paving, (especially for small channels) this form of bank paving is satisfactory from an engineering point of view.

Timber and concrete cribs are sometimes used for bulkheads and retaining walls to hold steep embankments, particularly where lateral encroachment into the river must be limited. Cribs are made up by interlocking pieces together in the manner shown in Figure 6-9. The crib may be slanted, or vertical depending on height and the crib is filled with rock or earth. Reinforced concrete retaining walls are alternatives to timber cribs which can be considered. However, concrete retaining walls are expensive and are generally only used in special confined locations where space precludes other methods of bank protection. In constructing concrete retaining walls drainage holes (weep holes) must be provided. The foundation of these walls should be placed below expected scour depths.

Retards are permeable devices placed parallel to embankments and river banks to decrease the stream velocities and prevent erosion (Figure 6-2). The design of timber pile and rockfill retards is essentially the same for timber and rock dikes discussed in previous

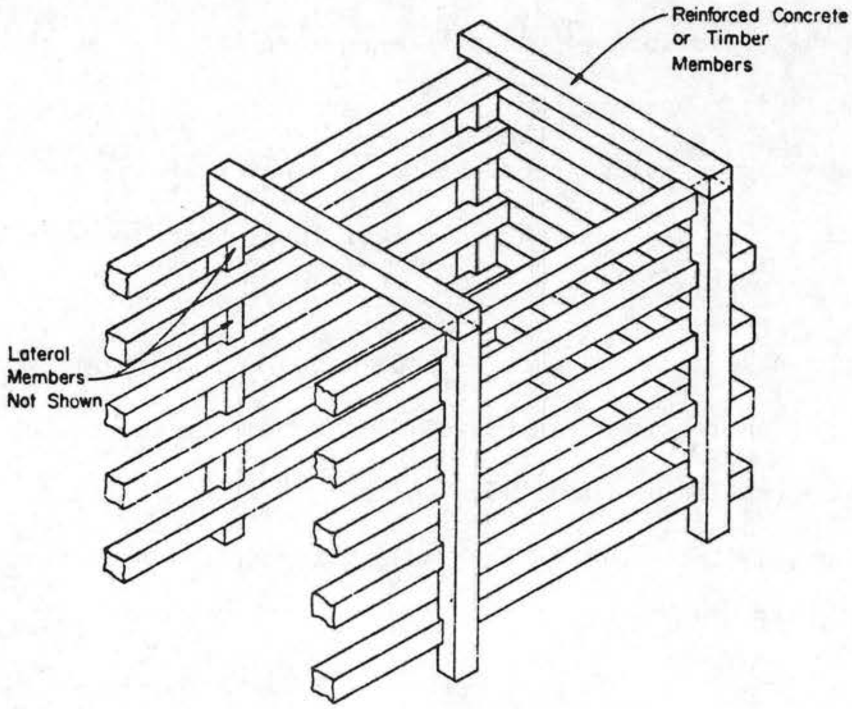


Figure 6-9 Concrete or Timber Cribs.

paragraphs. Retards may be used in combination with bank protection works such as riprap. The retard then serves to reduce the velocities sufficiently so that the riprap behind the retard is stable.

6.2.4 Guide to Channel Improvement, River Training, and Bank Stabilization

The type of channel improvement and devices used for training and bank stabilization depend upon the size of river with regard to width, depth and discharge; type of river, that is, meandering, braided or straight; sediment transport in terms of concentration and size distribution; length of river to be protected; availability of materials; environmental considerations; aesthetics; legal aspects; river use with regard to navigation, recreation, agriculture, municipal and industrial purposes; and perhaps other factors.

Table 6-1 is presented as a guide to assist in considering alternatives and formulating decisions for channel improvement and selection of type of bank protection and river training works. A river is first categorized as to size and type. The descriptors large, medium and small are relative terms but should give no interpretive problem. Straight rivers are those which have sinuosity less than 1.5. In general long reaches between meander bends which are essentially straight may be included in the straight river classification. Because these straight reaches are part of meander systems, stabilization and improvements may be required. The X in the box indicates that consideration could be given to use of the particular device. The absence of a check mark in the box indicates that the devices are not often used, but consideration could be given to them in special circumstances. Additional remarks are noted on the table.

6.2.5 Channel Stabilization Applications

As Henley (1966) notes in a review of channel stabilization work in the Vicksburg District on the Lower Mississippi River,

> "It has been found that stabilization of the river in this district cannot be solved by the application of hydraulic formulae alone, nor by using techniques and designs alone that have proved successful in the past. Past efforts, instead, indicate that there is a need for a mixture of hydraulic engineering, experience, rules of thumb, and old-fashioned American ingenuity to successfully control this river. (In many cases all of these elements have been used to no avail, and total and unexplainable failure has occurred.)"

If it could be assured that the high- and low-water channels coincide, planning for a stabilized channel suitable for year-round navigation would not constitute a major problem. However, it is very difficult, and in most cases impracticable, to design and properly place regulatory works so that they will be completely effective at all

Table 6-1 Guide for Selection of Methods and Devices for River Channel Improvement and Bank Protection Works

Size of River	Type of River	Channel Improvement	Dikes			Retards		Jetties		Bank Protection					
												Mattresses			
			Timber	Stone-fill	Earth	Timber	Steel Jacks	Timber	Steel Jacks	Riprap	Rock Trench	Rock and Wire	Concrete	Other	Cribs
Large	Meandering		X	X	*	*		X		X	X	*	X	X	
	Braided	X	X	X	*	*		X		X	X	*	X	X	
	Straight		X	X		*		X		X	X	*		X	
Medium	Meandering	X	X	X	X	X	X	X	X	X	X	X	X	X	X
	Braided	X	X	X	X	X	X	X	X	X	X	X	X	X	X
	Straight	X	X			X		X		X	X	†		X	X
Small	Meandering	X				X	X	X	X	X	X	†		X	X
	Braided	X				X	X	X	X	X	X	†		X	X
	Straight	X								X	X	†		X	X

* Floodplain embankment protection

† Where large rocks for riprap are not available

stages where the fluctuation in stage may cover a range of 50 feet or more. Since the river level is constantly changing, such hydraulic factors as current velocity, current direction, cross section, and sediment load also change.

Figure 6-10 is an illustration of typical stabilized bends, showing the flow trace for high- and low-water stages. In planning a stabilized channel, the objective is to mold and shape the river into such an alignment that the energy of the river can be harnessed to produce the desired channel. In designing such a channel, an attempt is made to fix the low-water flow trace so that after passage of floodflows the deepest thread of this trace will return to its previous position. During high river stages, when the point bars and smaller islands are submerged, the flow trace of the river channel may be a half mile or more from the low-water flow trace.

Major channel stabilization problems generally fall into two broad categories: first, and most complex, is the relatively long, straight reach. Long, straight reaches present a difficult problem since they are very unstable and any local agitation can, and usually does, change the channel from one bank to the other. The problem here becomes one of judgment, deciding just where the channel should be placed and then, by the use of revetments, dikes, dredging, and training structures, molding and shaping the river into the desired alignment.

The second broad category is the shorter, more sinuous reach with divided flow conditions. Where such divided flow conditions exist there is again the problem of just where the channel should be, since there are two or more existing channels, it must be decided which will be the best from the standpoint of stability and navigation. Once

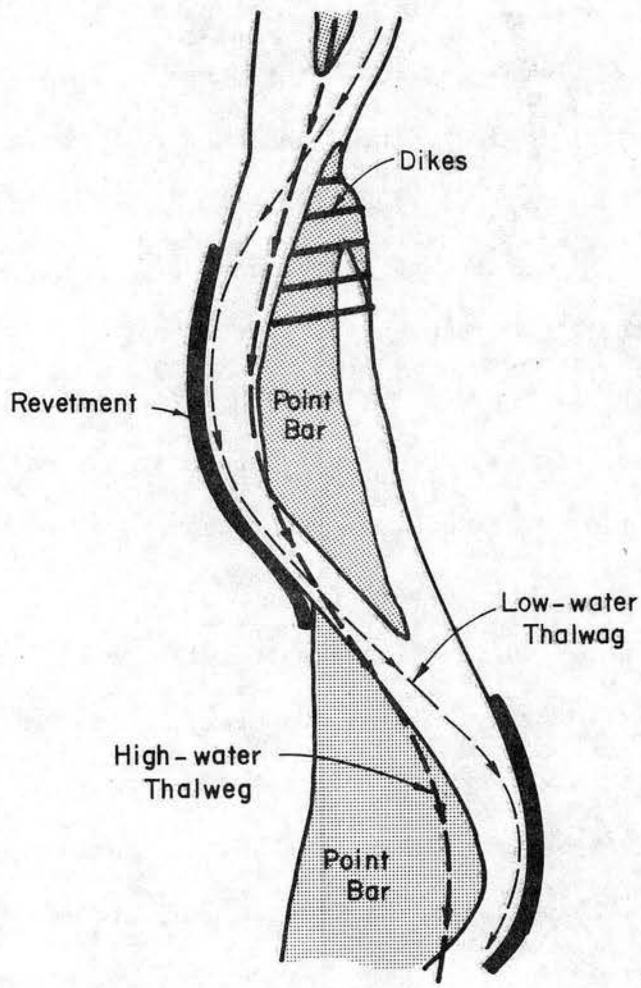


Figure 6-10 Typical Stabilized Bend of the Mississippi River, Showing the Flow Trace for High- and Low-Water Stages.

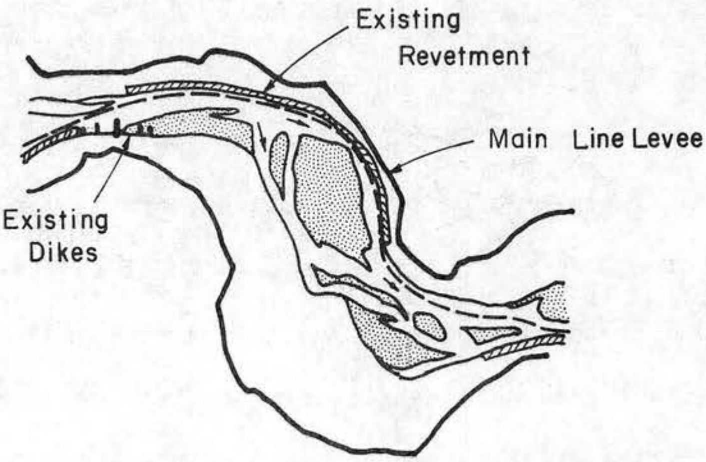
this design is made all efforts can be devoted to confining the river to that channel.

With the available stabilization tools in mind, their use in the proposed treatment of a typical channel stabilization problem can be illustrated. Figure 6-11a shows a reach of the river in which divided flow conditions have caused trouble for many years. In this area all the available channel stabilization tools have already been or will be applied. The existing revetments were placed for protection of main-line levees. They have been extended, repaired, and reinforced as necessary to continue their effectiveness. The pile dikes, at the upper left, were constructed to shut off a secondary channel to keep the main channel well upstream on the revetment across the river.

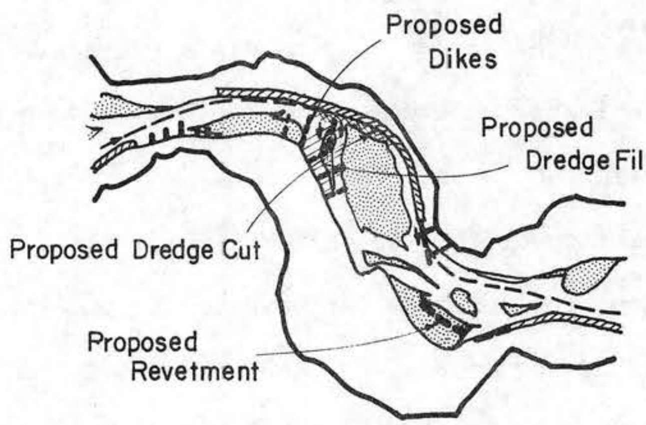
Figure 6-11b shows the proposed work which will be required for final stabilization of this reach. The heavy dashed lines indicate standard articulated concrete mattress revetment, the dashed lines indicate training structures, the double hatched area indicates required dredging, and the single hatched area indicates dredge material disposal.

The first step toward improvement for navigation will be to encourage bendway flow. This will be accomplished by: (a) closing any gaps between the existing revetments, which will cause deepening along the left bank; and (b) development dredging along the desired alignment. The dredged material will be used to deteriorate the pointway channel.

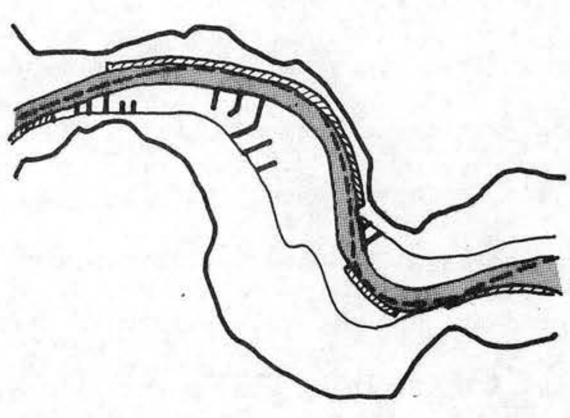
When the bendway channel has developed sufficiently to ensure year-round navigation, training and contraction works will be constructed to close off the pointway channel and direct flows into the bendway channel. Development dredging will be continued as required to improve the



(a) Short Reach with Divided Flow



(b) Divided Flow Reach with Revetment Dikes, and Dredging to Stabilize Channel



(c) Predicted Stabilized Channel for Reach Shown in Fig. (a)

Figure 6-11 Stabilization of a Divided Flow Reach.

bendway channel, with dredged material being placed in the training or contraction works.

As the pointway channel deteriorates and the preponderance of flow shifts to the bendway, the crossing at the lower end of the bendway should naturally readjust itself into the desired alignment along the proposed revetment. The training structure in the crossing will be constructed as necessary to establish and hold the desired alignment. As the desired alignment develops, the upstream extension of the revetment on the right bank will be constructed.

Figure 6-11c shows the predicted results of the efforts toward channel improvement in this reach. The stabilized channel will protect levees, pass floodwaters efficiently, provide good alignment for navigation, and will require minimum maintenance dredging to provide project depths. It should be emphasized here that this represents a typical hydraulic engineering solution to the stabilization problem. No attempt has been made to illustrate the far more complex problem posed by a requirement to satisfy both engineering and environmental considerations. Such a solution would, of necessity, require a compromise between the degree of stabilization achieved and the extent of environmental impact considered acceptable.

6.3 Evolution of Man-Made Side Channels

The morphology of divided flow reaches and the evolution of islands and natural side channels in the river environment have been examined in Section 3.5. 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 (see Figure 5-13 for example). As a result many of the natural side channels have been closed and 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 detailed examination of the effect of contraction dikes on river morphology and hydraulics is presented in the next section.

Simons, Schumm, and Stevens (1974) conducted a detailed study of the relationship between dike fields and side channels based on experiments in a laboratory flume and on field study of dike fields in the Middle Mississippi River. The results of this study constitute the best available description of the influence of contraction works on the development and evolution islands and side channels.

6.3.1 Laboratory Analysis

The contraction of the Middle Mississippi River with dikes has eliminated most of the natural side channels. In many cases, these natural side channels have been replaced with new man-made side channel In most instances it is beneficial to the ecology of the river to retai and maintain man-made side channels as well as the natural side channel The main problem is that the life of a side channel produced by dike fields is usually relatively short. The dike fields and the side chann fill with sediment rapidly because dike fields are usually located in areas of natural deposition. Once the side channel is filled with sediment, man has easy access to the island area. In many cases, the fille side channel area and island area are converted to agricultural use.

The life history of side channels and dike fields is evident in al reaches of the Middle Mississippi River. In almost every reach, there are old dike fields completely covered by sediment and vegetation, whic are now undistinguishable from the mainland; there are new and old dike visible only where they cross backwater channels and at the main channel extremity; and there are new dike fields as yet not covered by sediments and vegetation. A side channel in a dike field passes through stages of development usually to a stage where the side channel is undistinguishable from the adjacent flood. This evolutionary process operates on a time scale of years or tens of years. However, a hydraulic model study in the laboratory offers a means of accelerating this process under conditions which permit observing the details of the evolutionary sequence.

6.3.1.1 The Single Dike. When a dike is projected out from one bank into the channel flow, the flow velocities are increased, especially around the nose of the dike. These increased velocities scour sand from the region around the nose of the dike. Because the bed velocities at the nose of the dike are still less than the surface velocities, it is the sediment-laden bed velocities that make the turn into the lee side of the dike. In the lee side of dikes placed in natural depositional areas, where the flow expands again, the sediments are deposited. Figure 6-12 is a photograph of a bar formed downstream of a dike projected two feet out from the bank of a six-foot wide, straight laboratory channel. The direction of flow is from the bottom to the top of the photograph. The texture of the sand on the bar identifies the bar and the direction of flow on the bar.

The scour hole at the nose of the dike in Figure 6-12 is much smaller in volume than the bar behind the dike. After the scour hole has reached an equilibrium depth, the flow field around the nose of the dike takes the normal bed materials moving in from above the dike and places these sediments on the bar. Thus in the model, the bar continues to grow, even after the scour hole has ceased growing.

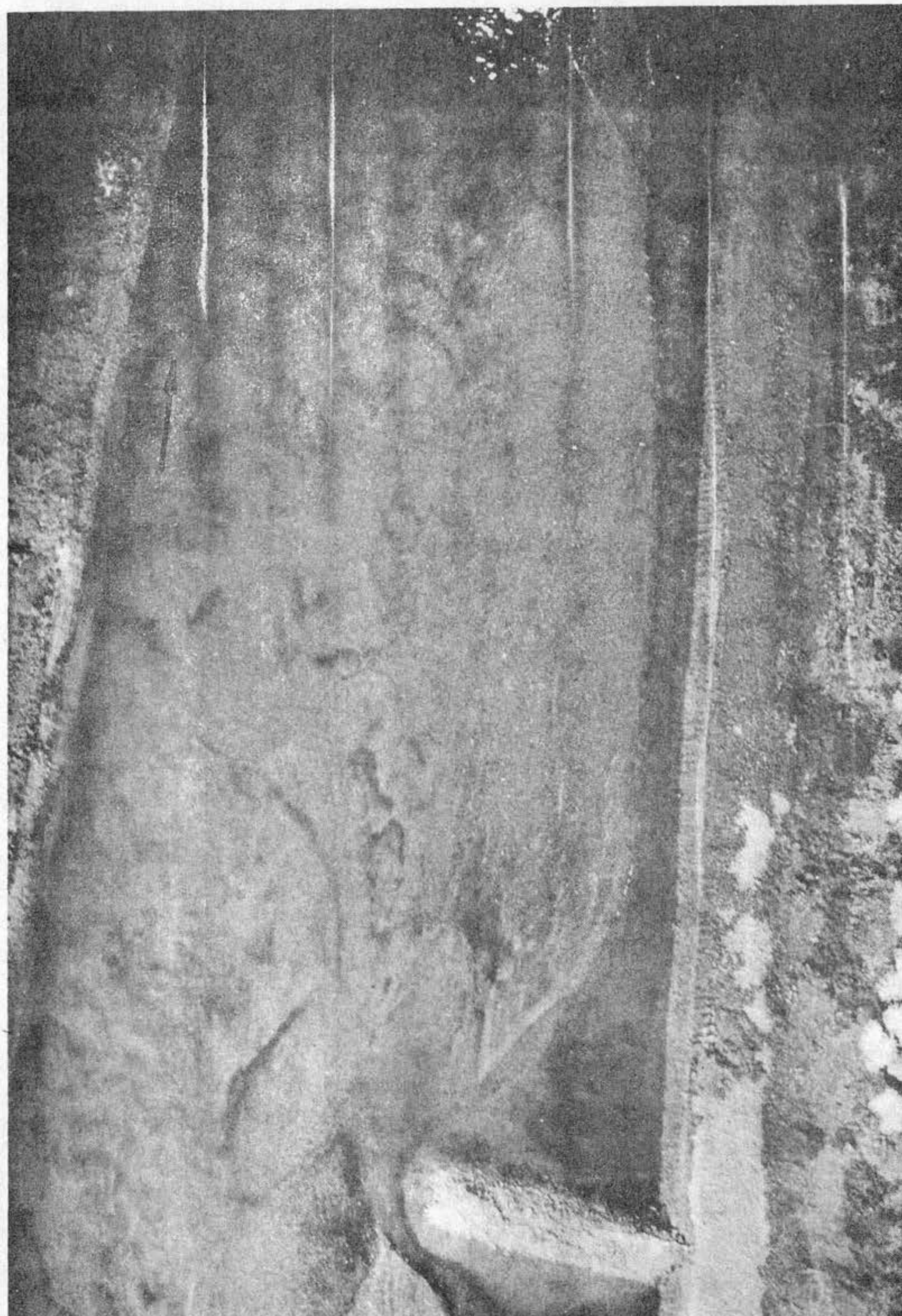


Figure 6-12 Bar Downstream of a Dike.

As soon as the bar is formed, a derelict channel is left in the area between the bar and the bankline. This channel accepts the flow over the bar and drains that flow out the lower end. If the bar were to become vegetated or otherwise stabilized, the small channel between the bar and the right bank (right side of the photograph) would become a side channel. The future of the side channel would depend on river alignment, discharge and sediment transport.

If left to evolve further, the bar shown in Figure 6-12 would continue to move into the area occupied by the small channel until the channel became obliterated. The channel would be closed by sand avalanching over the crest of the bar and into the channel. The upper part of the small channel would close first, and thereafter closure would progress downstream. A calm backwater area would remain on the immediate lee side of the dike. After development, the level of the bar surface would be nearly equal to the flow stage that produced the bar.

The crest of the dike shown in Figure 6-12 was constructed so that flood flows passed over the crest but low flows did not. The height of the dike has an important influence on the buildup of the bar. If the dike crest is at the same elevation as the bed, no bar forms. As we increase the elevation of the crest, more and more flow must pass around the nose. Once the crest elevation is greater than the high-water level, the crest elevation has no further incremental effect on the flow. A dike with a higher crest level produces a shorter bar more rapidly than the intermediate level dike shown in Figure 6-12. Also, with the high dike and favorable upstream depositional conditions, the small channel fills more rapidly.

The effect of reducing the flood level in the laboratory model is equivalent to that of increasing the dike crest levels. Lower flood levels result in shorter bars and in a more rapidly filling small channel.

6.3.1.2 <u>Dike Fields</u>. If we add another dike on the same side of the channel downstream of the single dike shown in Figure 6-12, the bar building processes change significantly. The first effect is that much less flow enters the region between the two dikes and more flow passes in the contracted section. The second dike blocks the discharge of the small channel along the bank. Less water and sediment enter into the region between the dikes. The net result is that the bar between the dikes grows and moves inward much more slowly than when there was only a single dike.

In the model, the addition of the second dike is usually sufficient to cause a general degradation of the channel opposite the dike field. The degradation causes a lowering of low-water levels which could leave the small channel dry during periods of low flow.

The evolution of the bars and channels within a dike field in a straight stretch of laboratory river is illustrated in Figure 6-13. Flow was from the bottom to the top of the photographs. The three model dikes were constructed so as to be submerged during floods but exposed at low flow.

The photograph in Figure 6-13a was taken after repeated hydrographs failed to move additional sand onto the bars between the dikes. The combination of flow over the dikes and blockage of the discharge in the side channel by the downstream dikes produced small bars and relatively large side channels in the dike field.

a. Lead dike crest is raised

b. Center dike is notched

c. Side channel is filled

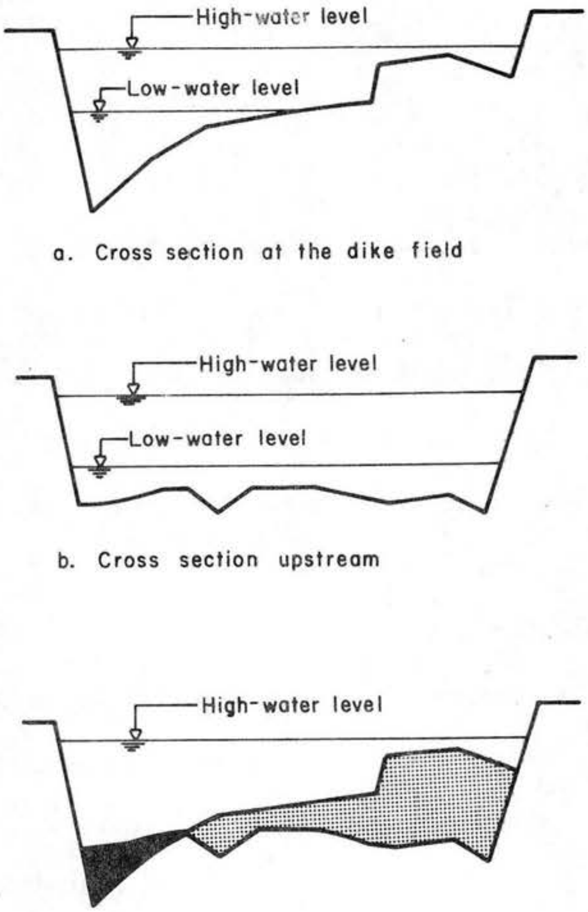
Figure 6-13 Evolution of Bars in a Dike Field.

In order to prolong the evolution of this dike field morphology, the elevation of the upstream dike crest was raised to a level above the flood level. The elevated lead dike is shown in place in Figure 6-13a, b, and c. With this modification the bars enlarged and moved bankward decreasing the size of the channel along the bank. The photograph in Figure 6-13a shows the change in bar size after the lead dike crest elevation was increased.

When the enlargement of the bar ceased before the channel along the bank filled, a notch was cut in the middle dike at the bankline (see Figure 6-13b). The notch permitted increased flow over the upstream bar which in turn built rapidly bankward. There was no increase in flow over the downstream bar, but the channel below the notch filled with sediment carried in from above. Soon the entire region in the dike field was filled with sediment as shown in Figure 6-13c.

The dike field performed admirably in terms of forming a deep, low water main channel. The cross sections of the main channel in the dike field reach and in the reach above the dike field are shown in Figure 6-14a and 6-14b. The vertical scale exaggeration in Figure 6-14 is five. The low-water channel was about twice as deep opposite the dike field as it was upstream of the dike field. There was no overbank flow in the model.

An overlay of the cross sections shown in Figures 6-14a and 6-14b is made in Figure 6-14c. The portion of the cross section which was degraded by the presence of the dike field is shaded dark grey. The depositional portion of the cross section between the dikes is shaded light grey. The cross-sectional area of deposition was much larger



c. Areas of erosion and deposition

Figure 6-14 Cross Sections at and above the Model Dike Field.

than the area scoured. The material in the deposition areas came from the low water channel and from that carried by the river into the contracted reach. Once the model dike field filled with sediment, the contraction had no measurable effect on sediment transport in the model.

In Figure 6-14a we can see that the dike field has narrowed the entire river channel except at stages above bankfull stage. The small channel and the surface of the bar were dry most of the time.

6.3.1.3 Vegetation. There is one important difference between the evolution which occurred in the laboratory model dike field described above, and the evolution which occurs in dike fields in the Middle Mississippi River. In the Mississippi River vegetation becomes established on the surface of bars and alters the evolution.

Figure 6-15 is a photograph of a vegetated bar immediately downstream of a single dike. This is the same bar as that shown in Figure 6-12. Immediately after the photograph in Figure 6-12 was taken small white plastic trees (shown in Figure 6-15) were added to the cres of the bar. The trees impeded the flow across the top of the bar and effectively stopped the movement of the sand into the small channel along the bankline. With the addition of trees, the bar became an island and the small channel became a side channel.

The addition of dike fields in the reaches of laboratory river upstream and downstream of the single dike resulted in a gradual degradation of the riverbed. As the bed degraded, the low-water stage in each succeeding hydrograph became lower and more of the island became exposed. As parts of the island became exposed, more trees were added; first the dark trees to the left of the white trees, and afterwards the white tipped trees along the left bank of the island. Thus, the trees reflect different stages of bar development.

In the case of the single dike, the addition of vegetation to the bar helped preserve the life of the side channel by stopping the movements of large amount of bed sediments over the bar. Sedimentation still occurred in the backwater channel but at a very much reduced rate The sedimentation resulted from the settling out of silts and clays carried in suspension. In the laboratory this layer of silt and clay in the backwater channel was perceptible by eye, but could not be measured.

6.3.1.4 High-Water Levels. If trees are placed on the bars in the dike field shown in Figure 6-13, there would be very little flow through the bar area. The reduction in cross-sectional area at bankful

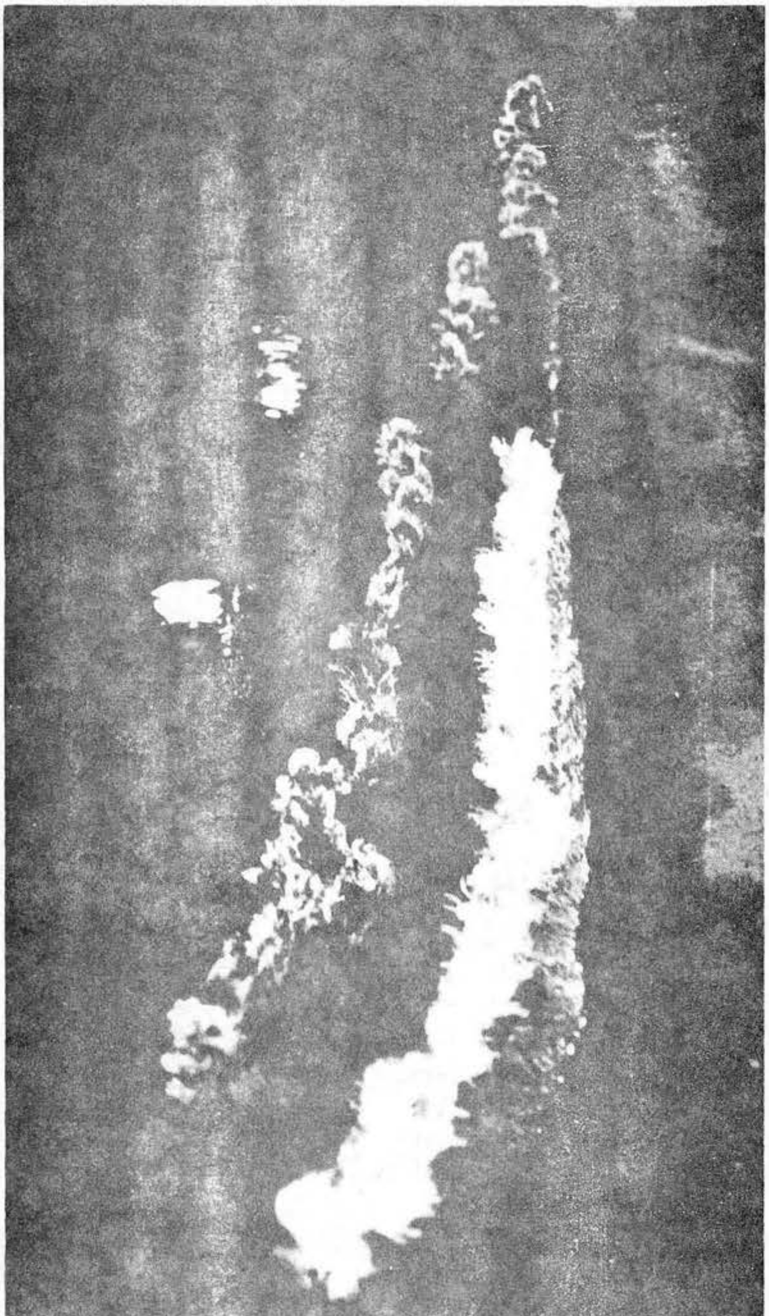


Figure 6-15 Vegetation on a Bar

stage caused by the dike field and the vegetation on the deposits in th dike field is illustrated in Figure 6-16. For example, in the laboratory model, the cross-sectional area at bankfull stage in the contracte reach was 1.1 square feet whereas in the natural channel upstream, the area was 1.8 square feet. Hence, for a given high-water discharge, stages are higher in the contracted reach than in the broad natural channel.

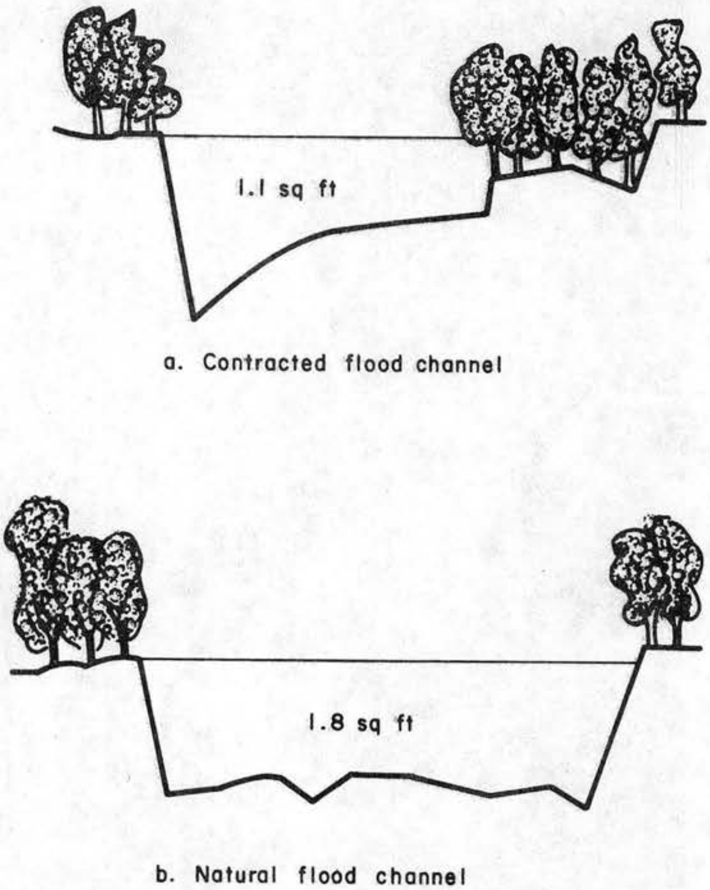


Figure 6-16 Comparison of Flood Channel Areas.

6.3.2 Observations in the Field

The features of the bars and the behavior of sediment and water in laboratory channels with dike fields and vegetation are essentially the same as those observed in the Middle Mississippi River. The photograph in Figure 6-17 illustrates the various stages of side channe development in many Middle Mississippi River dike fields. The long bar

Figure 6-17 Dike Fields in the Mississippi River.

at the bottom of the photograph is new; small dunes can be seen on its surface. The bar was formed by the extension of existing rock dikes. The two older pile dikes that extend only a fraction of the distance to the main channel are not effective. The channel between the bar and the vegetated island is wide and shallow and has a sand bed.

The backwater channel in the bottom left side of Figure 6-17 is a mature side channel formed by previous dike fields. The island between this old channel and the new channel is well vegetated. There are two trees growing on the dike that is still visible across the lower end of the backwater channel. Immediately downstream of this dike, we can see evidence in the vegetation that the downstream extension of this side channel has been completely filled with sediment and covered by vegetation.

The rate of sedimentation between the dikes is very rapid immediately after the dikes are built. The local scour at the nose of the dikes supplies the initial sediments; thereafter the bar building materials are bed sediments carried along by the river flows. When the bar becomes vegetated, sediment-laden bed currents no longer flow over the bar. Then, the sedimentation results from the settling out of the suspended sediments from the slow moving currents. The rate of sedimentation is reduced to about one to three feet per year. The major portion of this sediment is fine sand.

Later, the side channel becomes isolated from the main channel by a dense growth of vegetation. Very little water flows in the side channel because the path through the side channel is much more resistant to flow than in the main channel. The side channel has become a backwater channel. The bed and banks of this channel and the surface of the island are covered with mud which settles out from the slackwater that enters the side channel and covers the island during floods. This side channel is slowly deteriorating in size due to the yearly deposition of mud; the rate of deposition is on the order of one to five inches per year in the Middle Mississippi.

The ultimate fate of a mature side channel is obliteration by filling with sediments. The last phase is short in duration when vegetation can encroach on the side channel.

Once the side channels have filled, the plan view appearance of the river at all stages is the same as that of a straight and deep natural river. The hydraulics of this new river are different than before the introduction of dikes. This new river geometry is not considered permanent however. The experiences in the Middle Mississippi River are that the annual maintenance on dikes is substantial. This evolution is based on the maintenance associated with present day contraction and with the ice conditions and the modest flood flows that have occurred in the last two decades.

6.3.3 Conclusions

The evolution of islands and side channels on the natural river is discussed in Section 3.5.2. The analysis of historical changes in the river (Section 6.3.2) and results of geomorphic model studies (Section 6.3.1) have indicated the influence of contraction works such as dike fields and revetment on the evolution of side channels and on the river environment. These discussions support the following conclusions:

1. The 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. Future channel contractions will result in an increase in the depth of flow at all river stages.

3. Future channel contractions will decrease the river channel capacity at flood stage. The result will be higher flood stages for a given flood discharge.

4. Future channel contractions will lower the riverbed level and the low and intermediate water stages in the river. Stages will be lower on a greater number of days in the year. Lower stages affect groundwater levels in the aquifers connected to the river and affect tributary channels.

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

6. In the most part, future channel contractions by extensions of dikes will produce no new side channels.

7. Unless steps are taken to prevent it, ultimately nearly all natural and man-induced side channels should completely fill with sediment and become undistinguishable from the floodplain

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

9. Generally speaking, it is very difficult to design dike fields so that the resulting side channels will be self-maintaining. Dike fields are usually located in depositional areas of the river channel and suitable side channel intake positions are not available unless the flow is realigned upstream of the dik fields.

10. The life of a side channel can be increased greatly if the side channel can be isolated on the upper end from the main channel. When the side channel is isolated in this manner, the side channel is a backwater channel and the rate of sedimentation is very small.

11. Blocking an unsuitable upstream intake to a side channel will extend the life of that side channel. With the upstream intak blocked, the sediment supply to the side channel is reduced. Short side channels can be supplied with water during low stages from the lower end.

12. The notched dike may help in extending the life of a very few side channels. In general, the notched dike cannot be located in the proper position. Also, bankline instability will result where large scour holes occur next to the bankline.

6.4 River Response to Channel Stabilization

In this section a study of the geomorphic and hydraulic response of the Middle Mississippi River (St. Louis to Cairo) conducted by Simons, Schumm, and Stevens (1974) is reviewed to illustrate the response of an alluvial river to channel stabilization. River response to contraction is also examined using a mathematical model developed to determine the impact of dikes on the hydraulics and morphology of a river reach.

6.4.1 Development of the Middle Mississippi River

The objectives of developments along the Mississippi River have been to provide flood protection to people and property on the floodplain and to provide navigable depths for commercial transport during times of low-water flow.

In the 195-mile reach of river between the mouth of the Missouri River above St. Louis to the mouth of the Ohio River at Cairo, the Mississippi River is known as the Middle Mississippi (Figure 6-18). This reach is the hub of an interconnected inland river system serving the eastern, midwest and central plains regions of the United States.

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.

Figure 6-18 The Middle Mississippi River in the St. Louis District (after Degenhardt, 1973).

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.

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.

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 (see Section 6.2.3.1). 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 6-18) 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 riverbanks into the channel. The location and number of dikes in a 16-mile reach, which includes the prototype reach, are shown in Figure 6-19. In addition, 122 miles of bankline revetment to prevent bankline erosion have been placed in the Middle Mississippi.

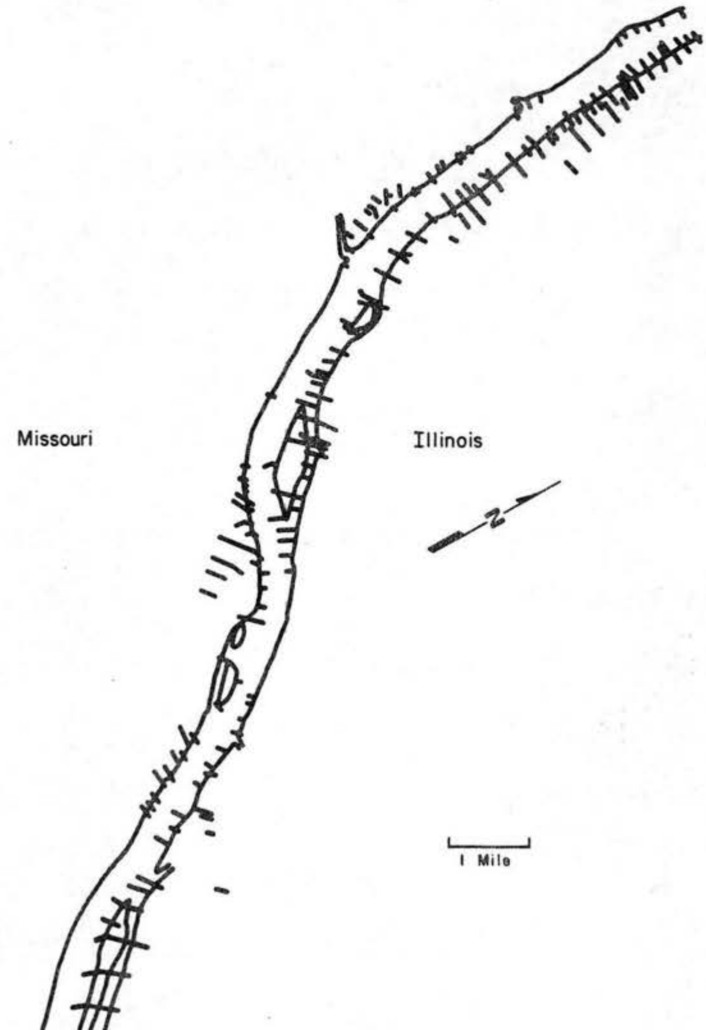


Figure 6-19 Dikes in 1970--RM 138 to 154 (after Degenhardt, 1973).

6.4.2 Geomorphic Response of the Middle Mississippi River

6.4.2.1 River Width. Using the same definition of river width as on the Upper Mississippi (Section 5.3.2.2), river widths on the Middle Mississippi were measured at mile increments between Jefferson Barracks and Cairo. The river widths are given in the following table:

Table 6-2

History of River Widths

Year	Average Width feet
1821	3600
1888	5300
1968	3200

The 1888 average width was approximately two thousand feet greater than the 1821 and 1968 widths, and the widths in 1821 and 1968 were nearly equal. There is evidence that the river did widen from natural causes in the period between 1800 and 1849. At the St. Louis harbor the following history of bankfull widths has been obtained:

Table 6-3

River Widths at St. Louis

Year	Width feet
1803	3100
1808	3200
1837	3700
1843	3900
1849	4200
1888	2100
1973	2100

Note: The widths for the period between 1803 and 1849 were obtained from Maher, 1963.

The river was definitely widening rapidly at St. Louis in the perio from 1803 to 1849. The widening was deteriorating the St. Louis harbor, and in 1838 the city and private corporations began work on a series of dikes from the Illinois shore to confine the river to a definite channel

The dikes reduced the riverwidth by half and since that time the bankfull width at St. Louis has remained 2100 feet.

It is possible that the large floods which occurred between 1844 and 1888, or a combination of large flood and floodplain development could have been the cause of the widening of the Middle Mississippi River reach. In that period there were four floods that equalled or exceeded 1,000,000 cfs.

6.4.2.2 The River in Cross Section. The Middle Mississippi River has been deepened for navigation by decreasing the width with rock and pile dikes. An example of the change of cross-sectional geometry is shown in Figure 6-20. In 1837 the river section at St. Louis was 3,700 feet wide and had an average depth of 30 feet deep at bankfull stage. The dikes started in the 1830's and completed before 1888

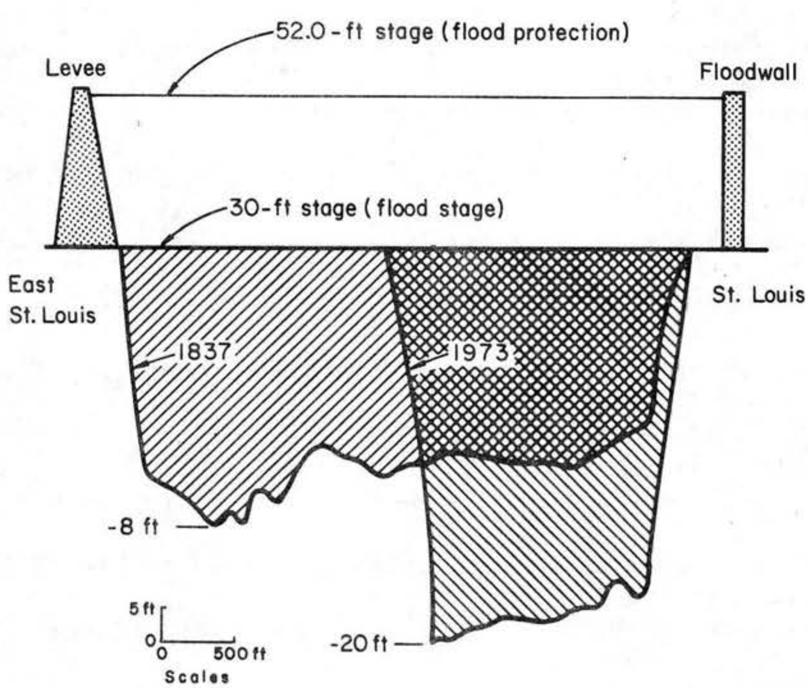


Figure 6-20 The St. Louis Cross Section.

decreased the width permanently to 2,100 feet. In 1973 the average depth prior to the 1973 flood was about 45 feet at bankfull stage and th width-to-depth ratio has decreased from 123 to 47. The river has been maintained in the narrow channel at St. Louis by protecting the undiked bank with revetment in places where bank erosion would occur.

The cross-sectional area at bankfull stage is approximately 80,000 square feet in 1973 whereas the area was 120,000 square feet in 1837. The narrowing of the channel at St. Louis has reduced the bankful channel area by about one-third. A similar decrease in the bankfull cross-sectional area has occurred throughout the Middle Mississippi rive wherever the river channel has been contracted.

6.4.2.3 The Longitudinal Profile. In the previous section, we compared the cross sections of the river channel at St. Louis before and after river contraction (see Figure 6-20). Narrowing the river section at St. Louis caused a general degradation of the bed. The bed was on the average 8 feet lower after contraction than before.

Riverbed degradation has occurred along the Middle Mississippi River wherever the river channel has been narrowed. The degradation is the natural consequence of reducing the width, increasing the flow per unit of width and increasing the transport capability of the water per unit width.

The riverbed elevations in a 14-mile reach of river are shown in Figure 6-21. The average bed elevation, shown in Figure 6-21, is the mean elevation of the riverbed in the low-water channel. The average be elevation was determined as the average of between 10 and 20 riverbed elevations at a cross section. The riverbed elevation is not necessaril

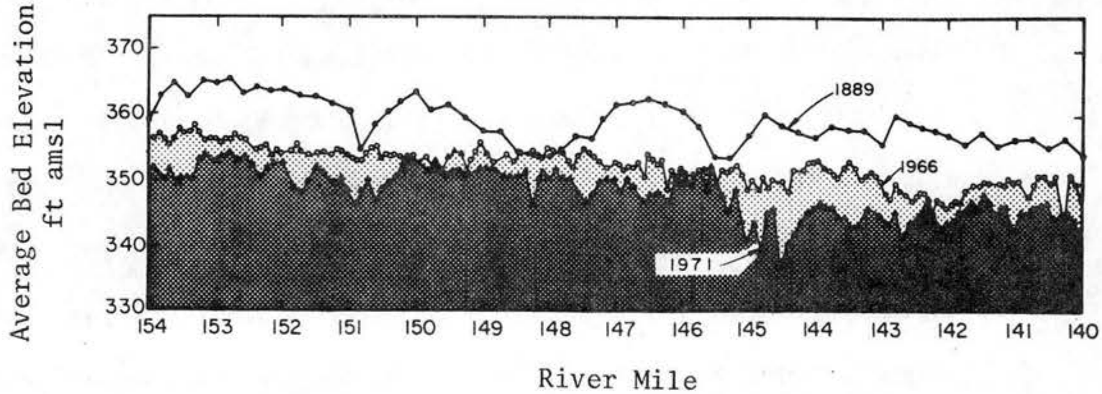


Figure 6-21 Bed Elevations--RM 140 to 154 (after Degenhardt, 1973).

related to depth of flow but is an indicator of degradation or aggradation in the river.

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. In July 1967, the Corps of Engineers selected this 14-mile reach as a test reach to develop design criteria on obtaining and maintaining a dependable 9-foot deep navigation channel (Degenhardt, 1973). Between 1967 and 1969, this test reach narrowed from 1800 feet to 1200 feet in width. In 1971, the riverbed was resurveyed. The 1971 bed profile is shown in Figure 6-21. The contraction from 1800 feet to 1200 feet had resulted in a 3-foot lowering of the riverbed. In 1971 the low-water riverbed in the 14-mile reach between River Miles 140 and 154 was on the average 11 feet lower than 1889.

6.4.3 <u>Hydraulic Response of the Middle Mississippi River</u>

6.4.3.1 <u>The St. Louis Gage</u>. The water flows in the Middle Mississippi River have been measured at St. Louis intermittently from

1843 to 1861, and continuously since 1861. St. Louis is below the confluence of the Missouri and Upper Mississippi Rivers. The flood peak discharge of record at St. Louis was 1,300,000 cfs recorded in 1844. Th Missouri River contributed 900,000 cfs, the flood record in the lower Missouri. The largest recorded flood in the Upper Mississippi was 565,000 cfs at Alton, Illinois which occurred in 1851 and again in 1858. The minimum discharge at St. Louis was 18,000 cfs which occurred in 1863

The construction of levees along the floodplain was one of man's first influences to affect natural flows in the Middle Mississippi. The floodplain is a storage area for floodwaters when the river rises above bankfull stage. Also the floodplain provides some channel capacity to carry water on downstream. Hence, levees along the reach of river increase the flow discharges for discharges greater than bankfull stage. The increase in discharge results from the decrease in floodplain storag

Because the floodplain was not protected by levees in 1844, the peak discharge of 1,300,000 cfs during the flood that year passed St. Louis at a 41.3-foot stage. Now due to the construction of contractive works and levee systems, it is estimated that the same discharge would pass St. Louis at approximately a 52.0-foot stage. While the peak discharge stage is now some 10 feet higher under developed conditions, as opposed to natural conditions, rural and urbanized areas suffer less flood damage under developed conditions due to the flood protection provided by levees than with no levees.

About 1907, levee construction in the Middle Mississippi began in earnest because the financing of levees was shifted from private land ow ers to the government. Until this time, levees were not effective becau of inadequate engineering capabilities and inadequate financial resource

The next dominant factor to affect flows was the construction of storage dams on the Missouri River, but the first dam was not completed until 1940. The larger dams are Yellow Tail on the Yellowstone River and Fort Peck, Garrison, Oahe, Big Bend, Fort Randall and Gavins Point on the Missouri. The effects of these reservoirs on the flows depend on the method of operation. In general the reservoirs have the effect of decreasing the maximum flows and increasing the minimum flows.

The net effect of upstream developments on the flows in the Middle Mississippi River at St. Louis are reflected in the St. Louis discharge records which are illustrated in Figure 6-22. The effects are:

1. The average annual peak flood discharge has not changed much in 110 years. On the average the present-day peak floods are only slightly lower than previously.

2. Large flood flows are not occurring as frequently now as in the past. In the decade between 1850 and 1860 there were three flood peaks greater than 1,000,000 cfs. Flood flows in excess of 1,000,000 cfs have not occurred since 1903.

3. The mean annual discharge has not changed in 130 years.

4. The annual minimum flow has been increasing slightly during the 130 years of record.

In general, the conclusion is that storage reservoirs, levees, dikes, land use changes, and any climatic changes have, in aggregate, not significantly changed the average annual flow in the Middle Mississippi. In terms of flood control the effect is that very large and very small peak flood discharges were more common in the natural river than in the river today.

6.4.3.2 Discharge and Stage Trends. The St. Louis river stage records begin in 1843, are intermittent for the period up to 1861, and are continuous on a daily basis thereafter. The annual maximum and minimum stages on the Market Street gage are shown in Figure 6-23. The zero of the Market Street gage is 379.94 feet above mean sea level.

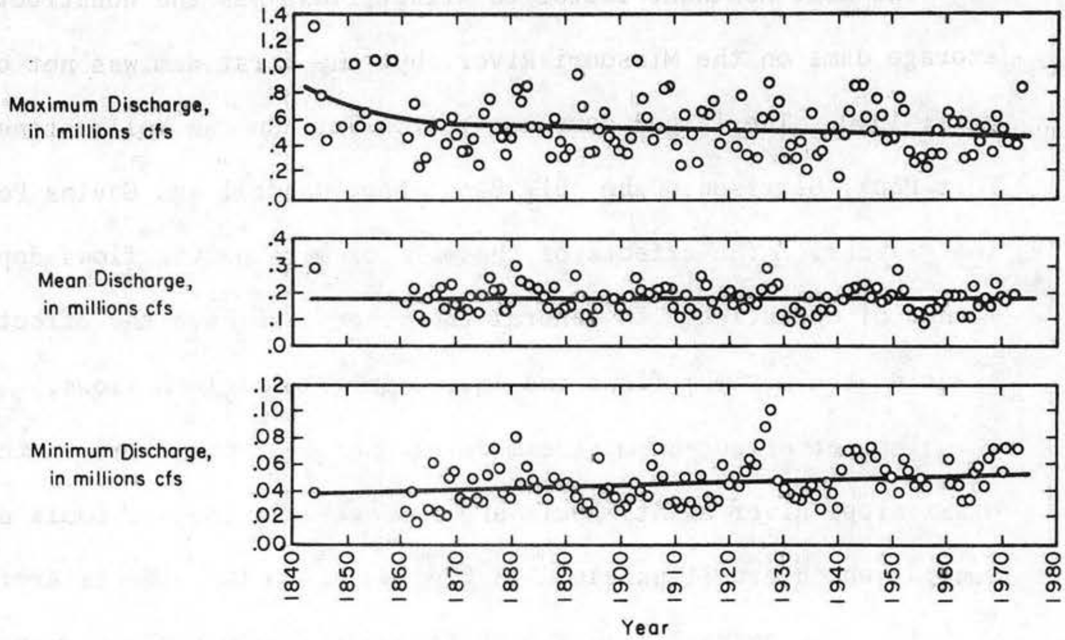


Figure 6-22 Discharges at St. Louis.

The annual maximum stage at St. Louis has been increasing only slightly throughout the 130 years of records. The variations in annual maximum stages are greater now than in the past. The highest recorded stage in St. Louis was 43.3 feet in 1973.

The trend of the annual minimum stages is downward during the peri of record (Figure 6-23). The minimum stages are now on the average 6 feet lower than in the 1860's and the 1870's. The lowest minimum sta at St. Louis was -6.2 feet on January 16, 1940.

The study of the daily stage versus duration curves reveal that, on the average, daily stages are lower now than a century ago. In the period 1861-1900 the stage equaled or exceeded 50 percent of the time was 11 feet whereas in the later period it was 2.5 feet lower. There have been more very low and very high stages in the last 70 years than in the first 40 years of record.

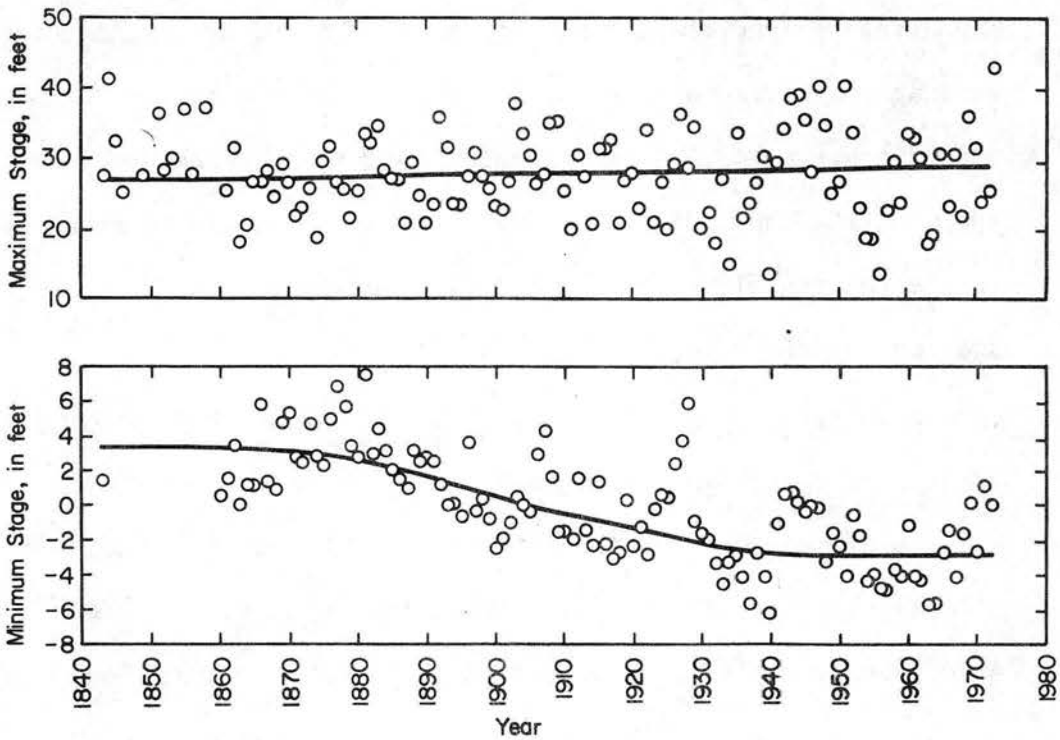


Figure 6-23 Stages at St. Louis.

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 6-4.

Table 6-4 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)

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 6-24. For example, the records show that the 1973 discharge of

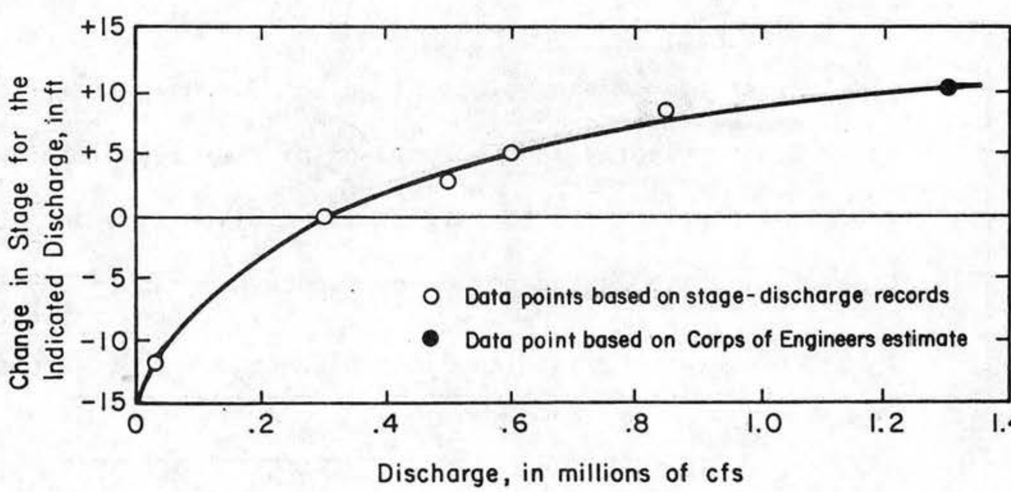


Figure 6-24 Changing Stages at St. Louis (after Simons et al., 1974).

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 6-24.

The estimate of the change in stage at 40,000 cfs was taken from Maher (1963). The increase in stage value at 1,300,000 cfs is the Corps of Engineers' estimate of the stage at St. Louis for the St. Louis district urban design flood (200-year flood). 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.

The changes in water stage at St. Louis in the last century can be contributed directly to the construction of rock and pile dikes and levees. Construction of rock and pile dikes cause deposition in the dike field, trees and willows grow on the deposit and stabilize the deposit. The tree and willow growth encourages additional deposition whenever the area is flooded. In most cases the ultimate effect of the dike field is to cause the river to develop a new bankline at the extremity of the dike field resulting in reduced channel width and a lowering of the riverbed level. The levees have isolated the major portion of the floodplain from the river channel so that all floodwaters are now confined to the river channel and that portion of the floodplain between the channel and the levees.

Because the bed is lower in the contracted river, the stages are lower than in the natural river most of the time. For a flow of 54,000 cfs, the stage was approximately 11 feet lower in 1946 than in 1837 (see Figure 6-25a).

The stage for which the 1837 and 1946 cross-sectional areas were equal was 19 feet. The discharge was 290,000 cfs for a 19-foot stage in 1946 and the area was 64,000 square feet (Figure 6-25b).

For discharges greater than approximately 300,000 cfs but less than 500,000 cfs, the increase in present-day river stage above the former

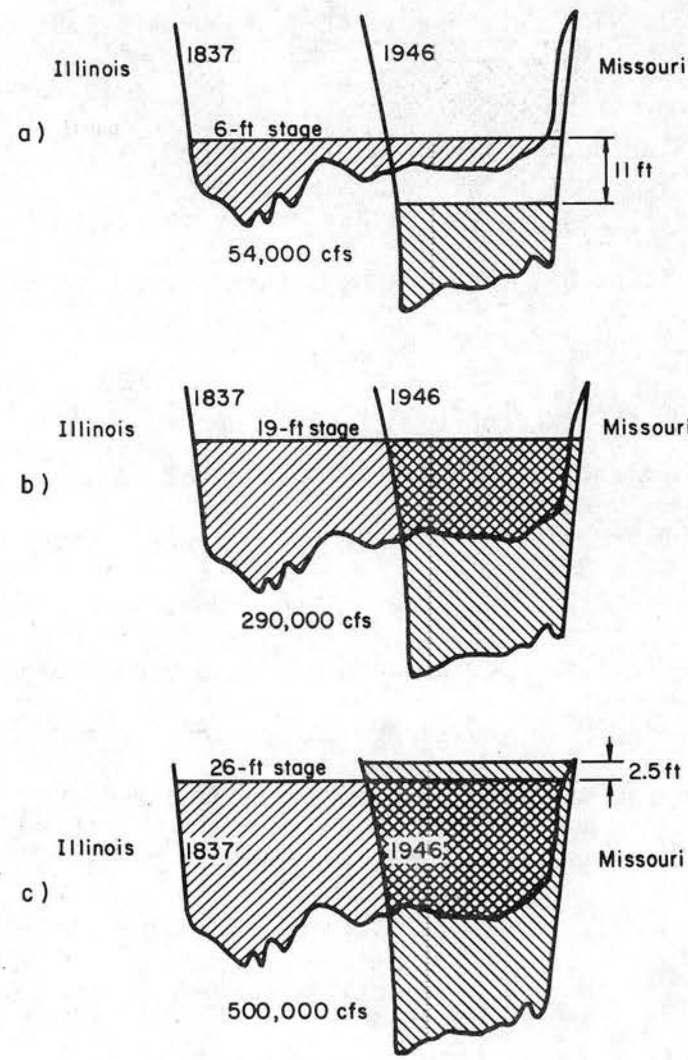
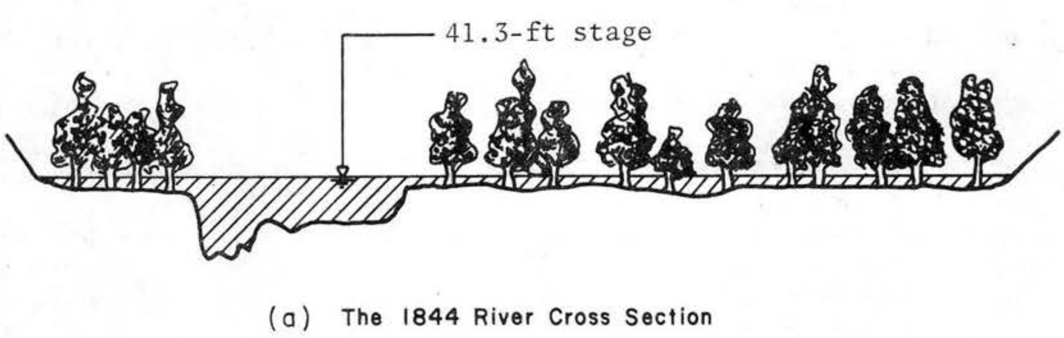


Figure 6-25 Flow Areas at St. Louis.

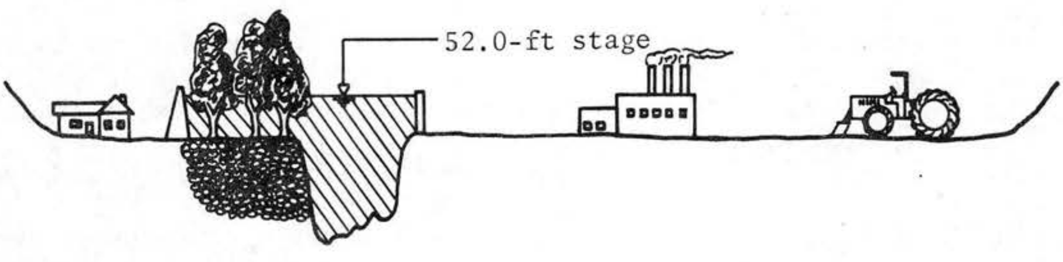
natural river stage is due solely to the dikes. Because the channel below bankfull stage is much narrower in the contracted river, the stage for a flow of 500,000 cfs is greater than in the natural river. For a flow of 500,000 cfs (the average annual peak flood) the cross-sectional areas occupied by the flow are shown in Figure 6-25c. At 500,000 cfs, the 1946 stage was 2.5 feet higher than the 1837 stage.

Once the river flows spill overbank, the levees as well as the dikes affect the high-water stages in the channel. For discharges slightly greater than bankfull, the affect of the levees on the stage is small. The former floodplain (now protected by the levees) was not efficient at carrying shallow flows. For larger floods, the floodplain carried more water than at lesser floods. The effect of the levees on the stages of these larger floods is more pronounced than at lesser floods.

Figure 6-26a illustrates a cross section of the Middle Mississippi River at St. Louis as it might have appeared during the 1844 flood of 1,300,000 cfs. The corresponding stage at St. Louis was 41.3 feet. Since completion of levee and dike construction in the Middle Mississippi River, there has been no flood of the magnitude of 1,300,000 cfs. According to the present-day assessment of the river behavior, the 1,300,000 cfs flood would pass St. Louis now with a high-water stage of 52.0 feet. The present-day cross section of the Middle Mississippi



(a) The 1844 River Cross Section



(b) The 1973 River Cross Section

Figure 6-26 The Cross Section for 1,300,000 cfs.

River at St. Louis would appear as shown in Figure 6-26b for a flood of 1,300,000 cfs.

The increase in river stage for any particular flood is the result of the combined effects of levees on the floodplain, dikes in the river channel and alterations of the floodplain between the levees and the river channel due to land use changes.

6.4.4 Mathematical Modeling of Response to Dikes

Many river response problems can be simulated with a one-dimensiona model for water and sediment routing as outlined in Section 2.4.4 and illustrated in Section 5.4. The primary assumption required for this type of mathematical model is that the channel to be modeled is sufficiently straight and uniform so that the flow characteristics may be physically represented by a one-dimensional model. There are, however, situations where the one-dimensional, single channel assumption does not provide the best representation of the physical configuration of the prototype reach. Configurations for which a multiple channel model is better adapted to represent prototype conditions include a divided flow reach and a braided reach. A multiple channel model also provides a better representation of response of a reach contracted by dikes along one bankline than could be achieved with a single channel model. The formulation of a multiple channel model and its application to the problem of river response to contraction works is described in this section.

6.4.4.1 The Multiple Channel Model. The morphology and hydraulics of the crossing and pool sequence of a meandering river have been discussed in detail in Section 3.4. It should be recalled that a change in stage in a meandering river is accompanied by a reversal of the processe of erosion and deposition on the crossings and in the pools of a meandering thalweg bendway. In general, at high stage the bendway pool scours and deposition occurs on the crossing; however, at low stages the crossing scours and the pool fills. In addition, the scale of these processes is significantly different at high and low stage. As sketched in Figure 3-16, at high flow the lower sediment transport rate on the crossing indicates a tendency to deposit, while the higher transport rate in the pool indicates a tendency to erode. Because of the higher flow velocities and greater volumes of water that accompany high-stage flow, the quantities of sediment that are both scoured and deposited greatly exceed quantities of sediment in motion at low-stage flow (Figure 3-16e). As a result low-flow scour on a crossing is generally not sufficient to remove the material deposited during high-stage flow. In the meandering thalweg system, then, the crossings act as sediment "source" areas and the pools act as sediment "sink" areas.

A hypothetical channel was developed by Dass (1975) at Colorado State University to approximate the patterns of erosion and deposition in a meandering stream. This channel simulates the high velocity, erosional environment of a bendway pool with the narrow contracted (600-foot) section shown in Figure 6-27. The lower velocity, depositional environment of the crossings above and below a typical bendway pool is simulated by the wider (1000-foot) sections shown in Figure 6-27. The crossing and pool sequence to be modeled is located in the central 6 miles of a 51.4-mile reach of river. To establish proper boundary conditions above and below this reach, the model includes a uniform 800-foot wide, 22.7-mile long reach upstream and downstream of this central 6-mile reach.

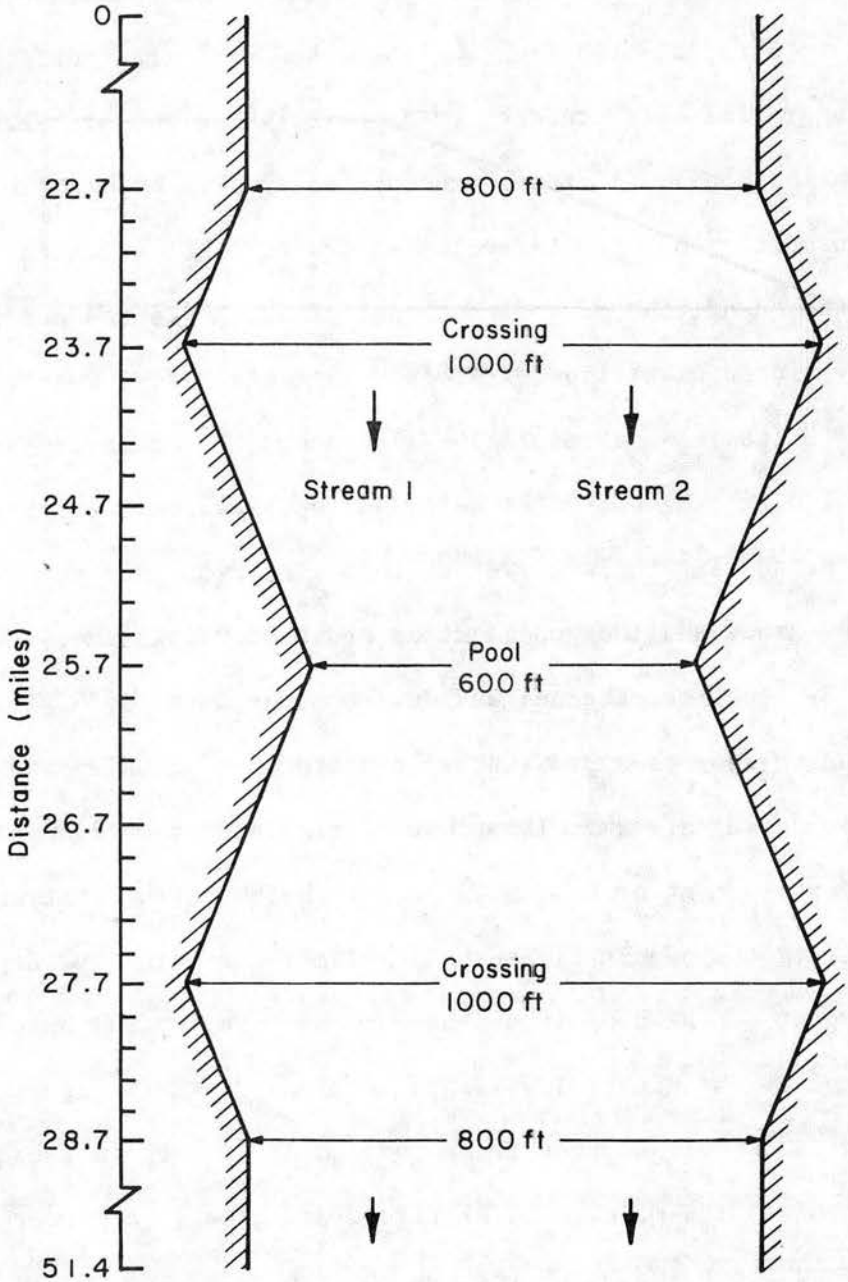


Figure 6-27 Hypothetical River Reach for the Multiple Channel Model.

Bed elevations along this hypothetical reach are varied to simulate natural conditions, with a linearly decreasing bed slope in the uniform channel sections, higher bed elevations (decreased water depth) over the crossings, and lower bed elevations (increased water depth) through the simulated bendway pool (Mile 25.7). This model incorporates the multiple channel concept by dividing the river reach into Stream 1 and Stream 2 along the channel centerline (Figure 6-27). The mathematical model routes both water and sediment through the two channels of the hypothetical reach and at the same time accounts for an interchange of water and sediment between the two streams by utilizing appropriate lateral inflow or outflow parameters.

For the hypothetical river reach shown in Figure 6-27 the following additional characteristics are assumed:

1. Manning's roughness coefficient, $n = 0.03$
2. The sediment volume ratio, $p = 0.6$, and
3. Total bed material discharge,

$$Q_s = a\ V^b, \tag{6.11}$$

where a and b are the coefficients which must be obtained by model calibration for the reach under consideration. For the hypothetical reach, it is assumed that

$a = 0.00001$, and

$b = 3.0$.

Water levels at the different sections along the reach are calculated based on an average water surface slope of 1 foot/mile and an initial discharge of 62200 cfs.

A stage hydrograph shown in Figure 6-28 is applied at the upstream end of the reach to simulate the routing of a typical flood through the modeled reach.

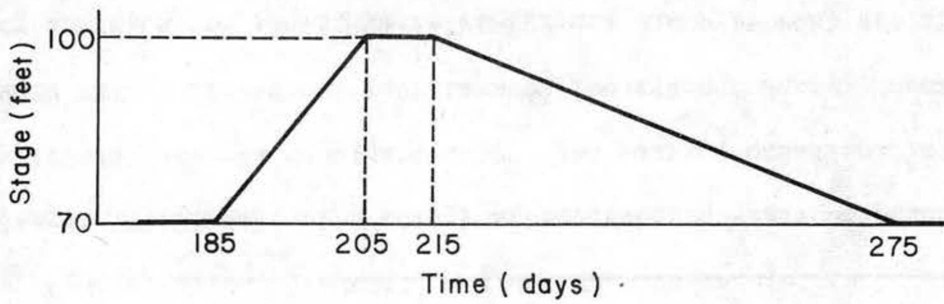


Figure 6-28 Stage Hydrograph for Flood Routing.

As a downstream boundary condition, the discharge versus depth rating curve below the central 6-mile reach is obtained by assuming the friction slope, S_f to be 1 foot/mile. For the channel width of 800 fee and Manning's n of 0.03, the discharge versus depth relation becomes

$$Q = 545.3\ y_o^{5/3} \tag{6.12}$$

6.4.4.2 Application of the Multiple Channel Model. In using the multiple channel model to evaluate response to modification of the assumed "natural" conditions of Figure 6-27, the response of the natural river must first be established. This is accomplished by routing the one year hydrograph of Figure 6-28 through the river reach under prescribed initial and boundary conditions. The bed elevation changes along the 6-mile crossing and pool sequence (x = 22.7 miles to x = 28.7 miles) of the reach is shown in Figure 6-29 for different time

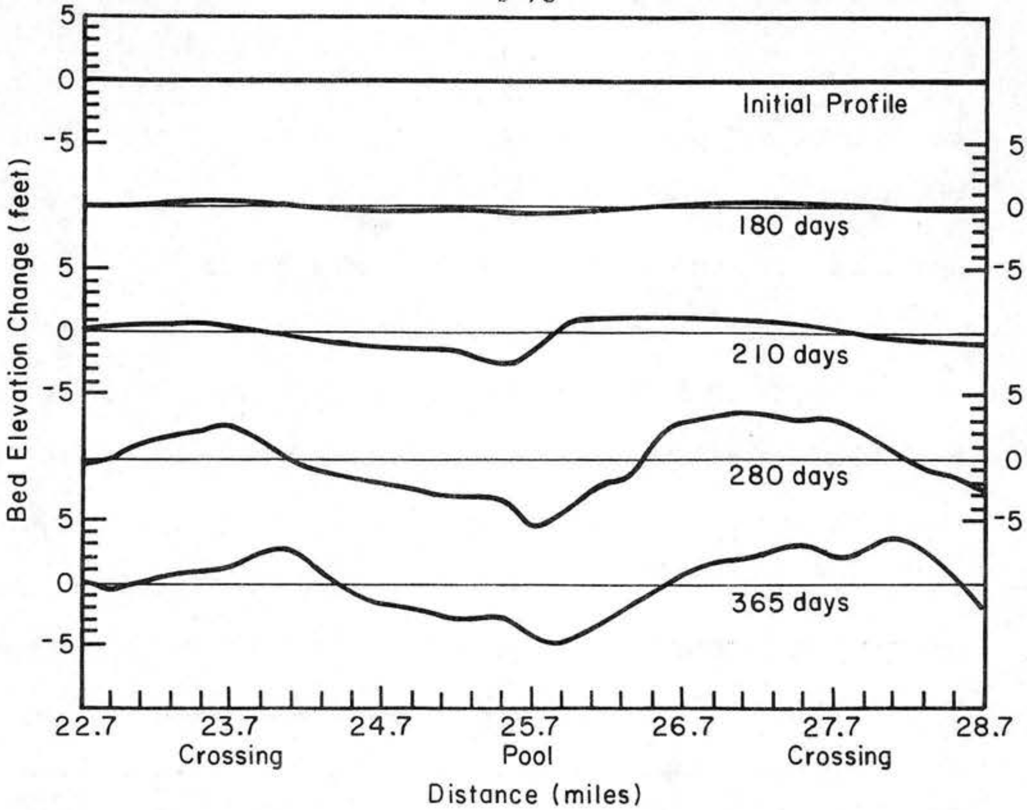


Figure 6-29 Profiles of the Calculated Bed Elevation Changes in the Six-Mile Crossing and Pool Sequence under Natural Conditions.

periods. These profiles indicate a general pattern of deposition at the crossing and erosion in the pool. At $t = 280$ days (during the low flow period), Figure 6-29 shows a deposition of about 3 ft at the upstream crossing ($x = 23.7$ miles), a deposition of about 4 ft at the downstream crossing ($x = 27.7$ miles) and an erosion of about 5 ft in the bendway pool. If it is assumed that this amount of deposition at the upstream crossing may impact the navigation channel in that section, it is apparent that dredging or contraction on the crossings will be required.

To test the feasibility of using contraction dikes to correct the adverse conditions created by deposition on the upstream crossing, the construction of a series of dikes along the left bank of the modeled reach can be simulated. This simulation is achieved by raising the bed elevation in Stream 2 at appropriate locations between Mile 22.9

and Mile 24.5 (upstream crossing). It is assumed that at each section the dikes occupy the entire half width of the channel at that section. The height of these dikes is varied along the reach.

Under these conditions, the same one year stage hydrograph (Figure 6-28) is routed through the channel reach. The sediment transport function and the boundary conditions are also kept the same. The change in bed elevations in Stream 1 (the right channel) under these conditions is shown in Figure 6-30. The desired scour on the upstream crossing in Stream 1 as a result of contraction dikes in Stream 2

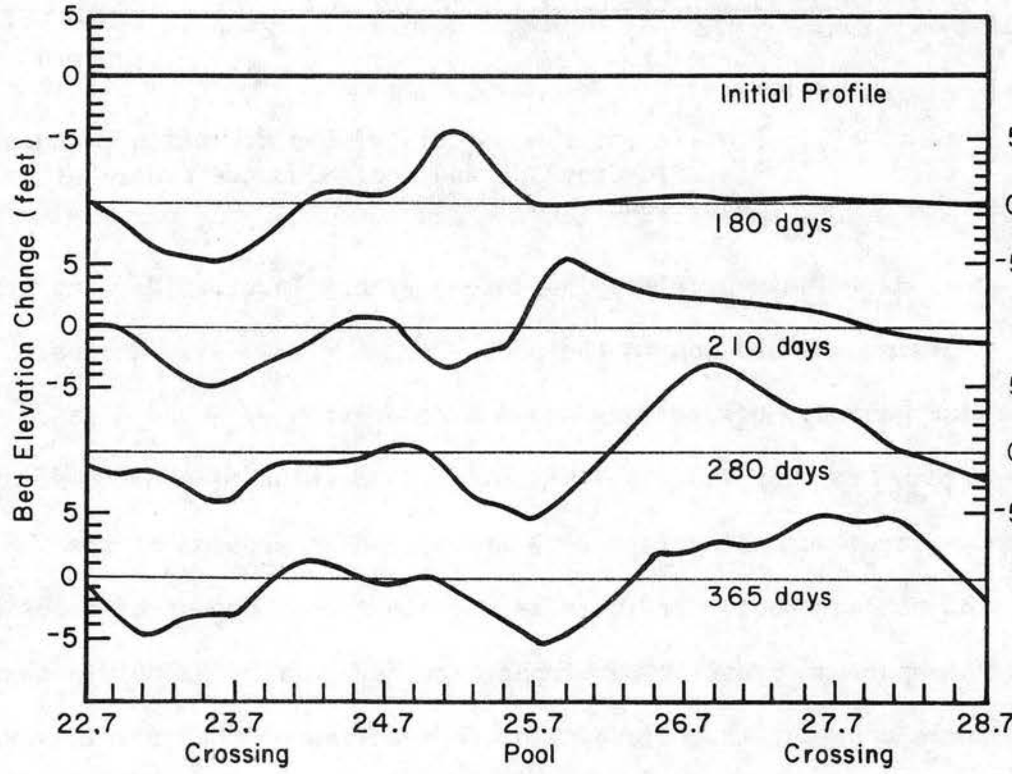


Figure 6-30 Profiles of the Calculated Bed Elevation Changes in Stream 1 of the Six-Mile Crossing and Pool Sequence after Simulating Construction of Contraction Works.

develops rapidly during the low flow portion of the hydrograph (180 day in Figure 6-30). The deposition of this material downstream from the crossing is also apparent. As the flood portion of the hydrograph passe through the reach (210 days), lowered bed elevations in Stream 1 at the upstream crossing are maintained. It is also apparent that the sand wave created by this scoured material moves progressively downstream through the reach. At 210 days (Figure 6-30) this sand wave has reached the pool at Mile 25.7. At 280 days the sand wave is approaching the downstream crossing, and after the reach has been subjected to the full year's hydrograph, the material scoured from the upstream crossing has been deposited over the downstream reach.

Bed elevation in Stream 1 after routing the one year hydrograph through the model reach under natural conditions and after simulating construction of contraction dikes in Stream 2 at the upstream crossing are compared in Figure 6-31. Significant deepening of the

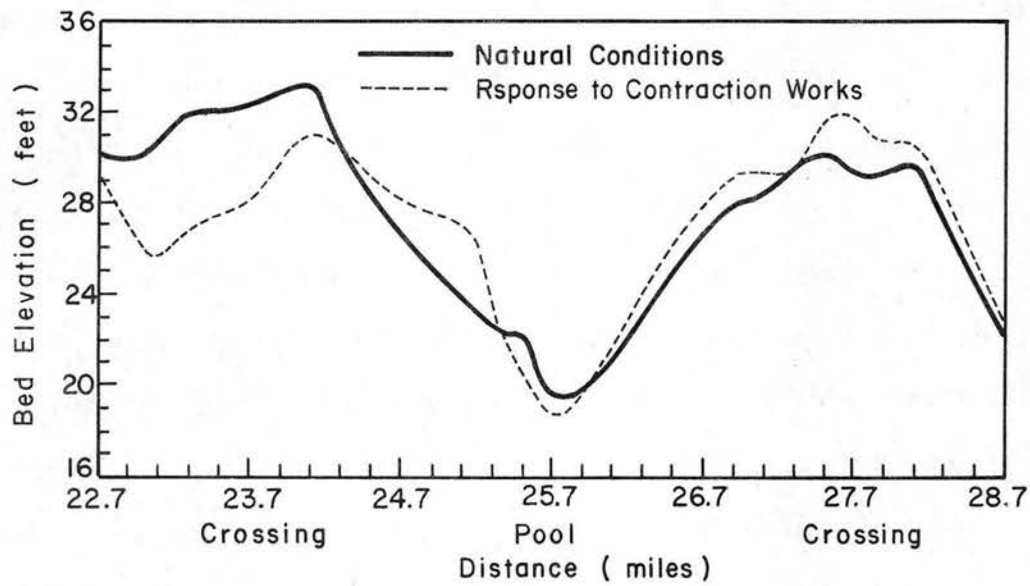


Figure 6-31 Comparison of the Calculated Bed Profiles in Stream 1 after Routing of the One-Year Hydrograph under Natural Conditions and after Simulating Construction of Contraction Works.

channel at the upstream crossing is apparent (Mile 23.7). Pool elevations (Mile 25.7) remained approximately the same, and a small increase in the deposition experienced under natural conditions at the downstream crossing (Mile 27.7) is apparent as a result of the sand wave that moved through the system.

The net result of constructing contraction dikes at the upstream crossing is to achieve the required deepening of the channel at that location. A period of instability following dike construction must be anticipated as material scoured adjacent to a dike field is redistributed through the system. In the reach below a dike field this instability should last only a few years. As indicated by Figure 6-31 the pool and crossing below the dike field have returned to nearly normal conditions after passage of a single year's hydrograph. With repeated applications of the annual hydrograph the perturbation created by construction of the dike field will continue to be distributed through the system, and the reach below the dike field will return to natural conditions.

It should also be anticipated that any change in the channel geometry of a river reach will create a change in the water surface elevations along the reach. If flood peaks are accompanied by an increase in the water surface elevation anywhere in the reach, it is quite likely that additional low-lying portions of the river valley will experience submergence. Any proposal to modify a river reach must be examined to insure that the new water surface profile along the reach does not at any time, exceed the accepted water surface elevation limit for the reach. The water surface profiles during the flood peak period for the natural and contracted river are plotted in Figure 6-32. It can be seen that as a result of constructing contraction dikes there is a gain of about 6 inches in stage in the upper portions of the modeled reach. Although a gain of this magnitude in the water surface elevation may be acceptable in view of the advantages gained from channel modification, the increased potential for inundating additional portions of the river valley must be examined and evaluated.

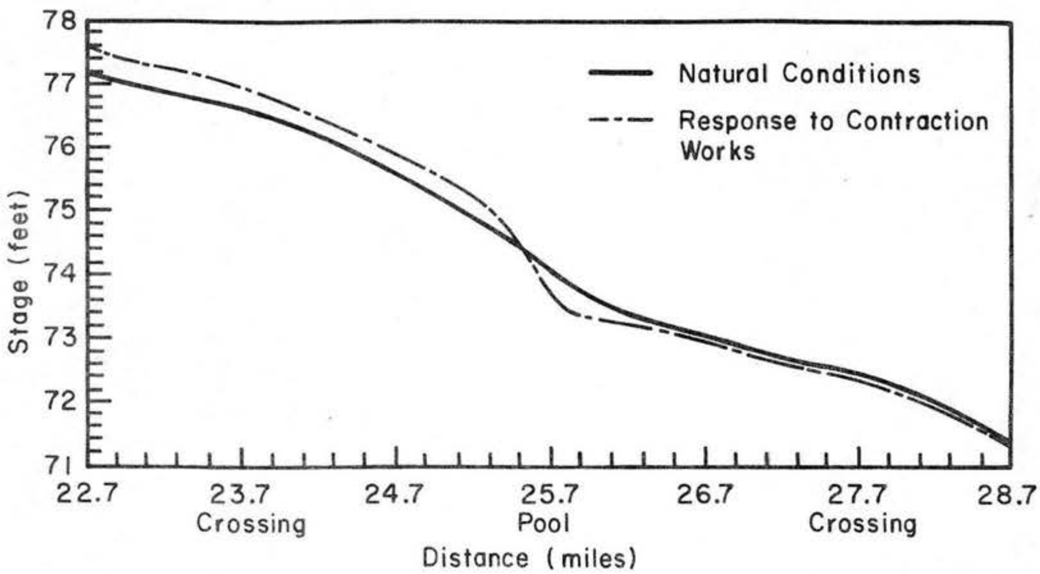


Figure 6-32 Comparison of the Calculated Water Surface Profiles at the Flood Peak (t = 210 days) under Natural Conditions and after Simulating Construction of Contraction Works.

Chapter 7

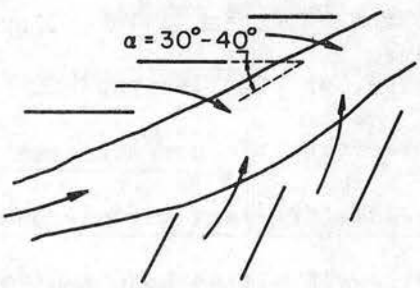
RIVER RESPONSE TO DREDGING

7.1 The Role of Dredging

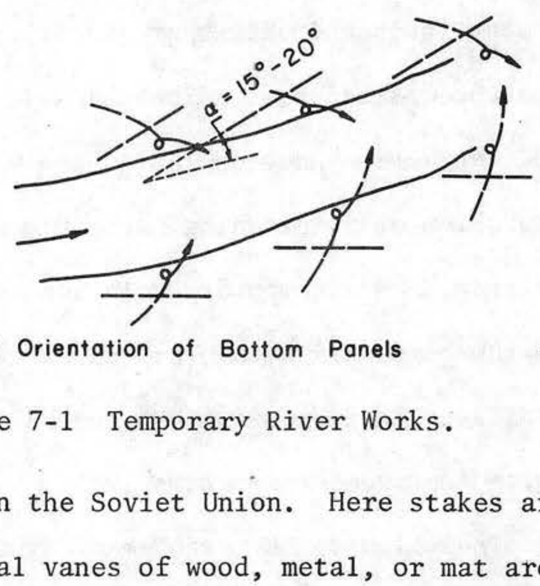
Dredging is defined as a process by which sediments are removed from the bottom of streams, lakes, and coastal waters, transported by ship, barge, or pipeline, and discharged in open water or on land. To the river engineer concerned with maintaining navigable depth in a waterway system, dredging is only 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 a solution to the problem of providing navigable depths in a waterway system, dredging can be compared to such temporary 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 degrees toward the main current and tend to direct the current into the main channel (Figure 7-1). Bottom panels have been



a) Orientation of Bandals



b) Orientation of Bottom Panels

Figure 7-1 Temporary River Works.

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 degrees into the flow (Figure 7-1). 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.

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 but 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 1930-31 dredging records of the Memphis District on the Lower Mississippi River illustrate the temporary nature of channel improvement by dredging. Although more than 73,000,000 cubic yards of material had been 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 increasingly greater draft and tonnage but no permanence of channel or assurance of adequate depths and widths had been obtained. The results were temporary in nature but effective in keeping the channel open.

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 Mississippi River 9-foot channel project and the problem of dredged material disposal are investigated.

7.2 Dredges and Dredging Operations

7.2.1 Dredge Development on the Mississippi River

The devices which have been used on the Mississippi River 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.

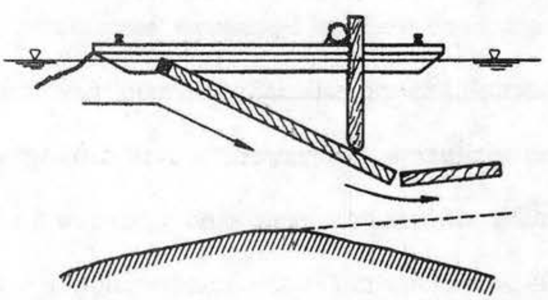
In the Transactions of the American Society of Civil Engineers, December 1898, J.A. Ockerson noted in regard to navigation on the Mississippi River: "The necessity for some suitable device for removal of sandbars has long been felt. Some thirty years ago a board of engineers recommended that a prize of $100,000 be offered for the best device for removing obstructing sandbars 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 reform 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." 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).

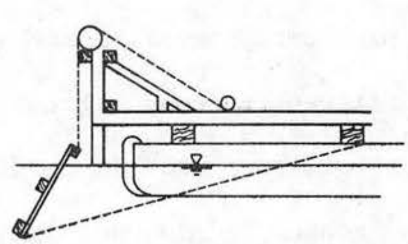
Among the first devices employed for dredging on the Mississippi were those that used a stirring or scraping technique. In 1867 $96,000 was appropriated to construct and operate two scrapers on the Upper Mississippi between St. Paul and the mouth of the Illinois. In operation, a dredge equipped with 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.

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).

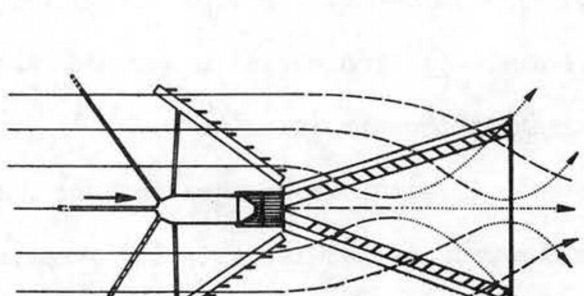
Numerous dredging devices that employ a current deflection principle have also been proposed. An example is the "Machine for Deepening River Channels," patented in 1880, whose purpose was to "deflect the current of a river downward, and thus cause the said current to deepen the channel." A typical deflector proposed for use on the Mississippi in the 1880's is shown in Figure 7-2a. In this



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 7-2 Current-Deflector Dredges.

same time frame, Ockerson (1898) reports that the "Kingston" deflector (Figure 7-2b) was successfully used on the Hooghly River in India where it cut from 0.5 to 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 7-2c) consists of a barge to which four vane systems are attached to redirect the flow and produce a spiral flow pattern behind 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.

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 66 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 improvemen The Suter-Flad committee was organized in 1891 and tasked to "investig 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, 19

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. 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, a water jets, harrows, plows, etc., trusting and expecting that the sand thus thrown up from the bottom will be carried off by the current." I 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. 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 at this time that a satisfactory type of dredge for Mississippi River channel maintenance had been developed.

7.2.2 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 7-3) 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

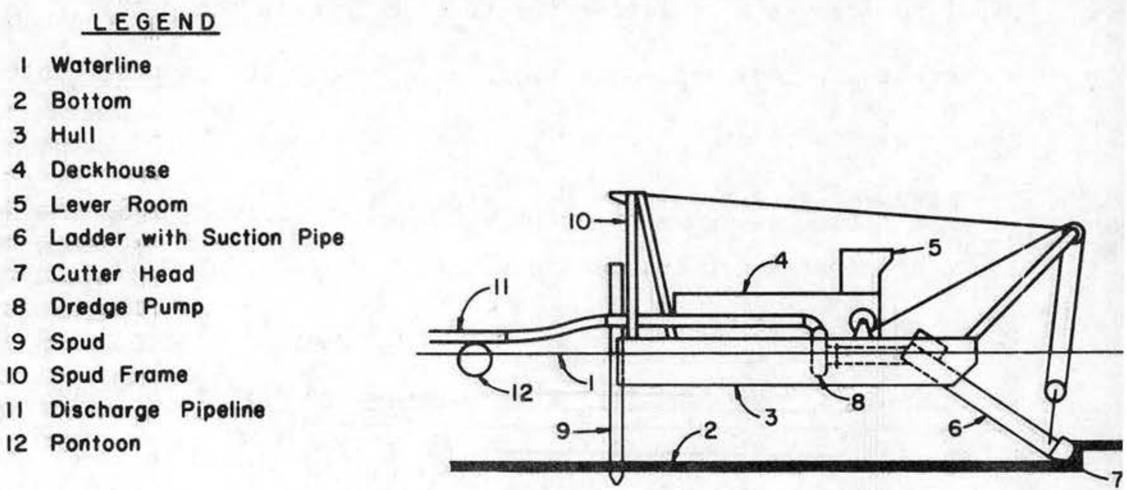


Figure 7-3 Cutterhead Dredge Components (after Black, 1973).

blades cut into subaqueous material which is then drawn into the suctic 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 cutter head 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 7-4). One

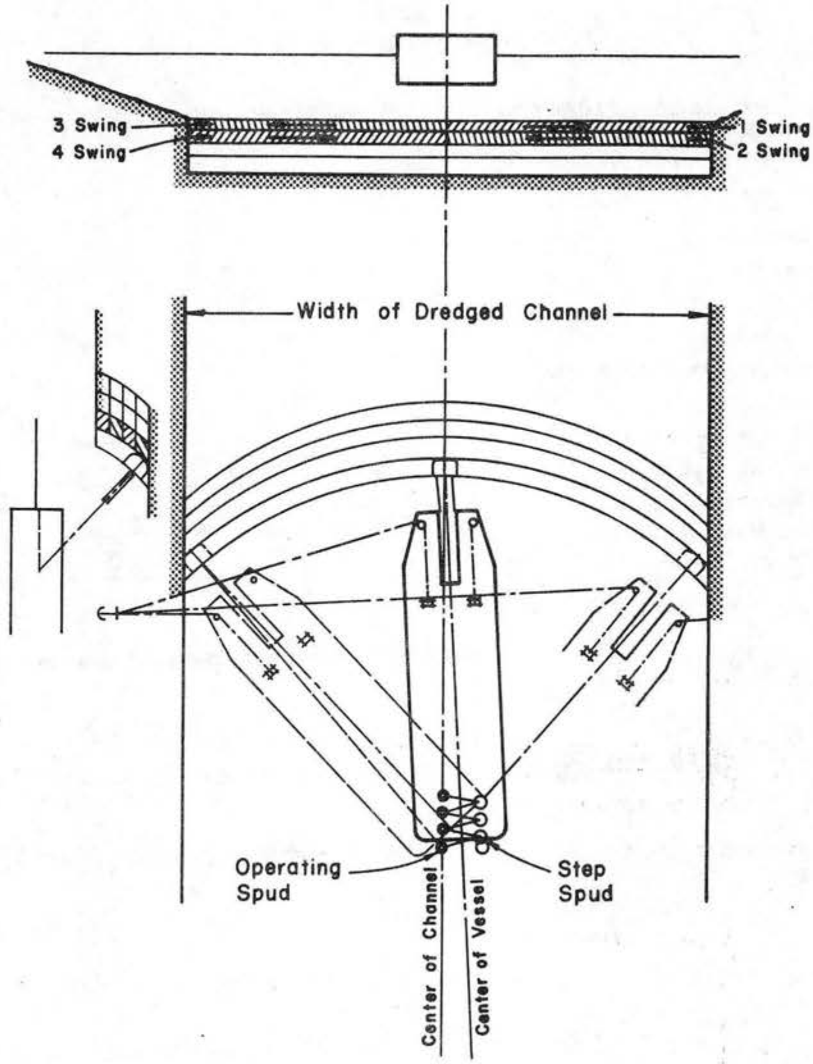


Figure 7-4 Operation of a Cutterhead Dredge (after NEDECO, 1965).

spud can be used as the operating spud and the other to step the dredge forward as in Figure 7-4, 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 7-4). 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 7-5).

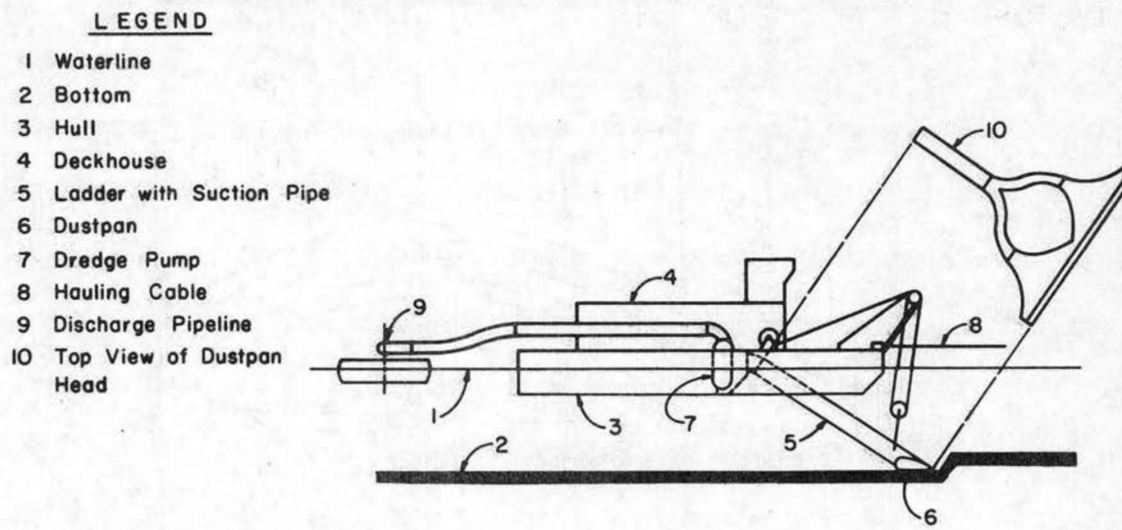


Figure 7-5 Dustpan Dredge Components (after Black, 1973).

It is best adapted to dredging relatively soft, easily eroded material and has been used extensively on the Lower and Middle Mississippi. Wit 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 7-6). 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,

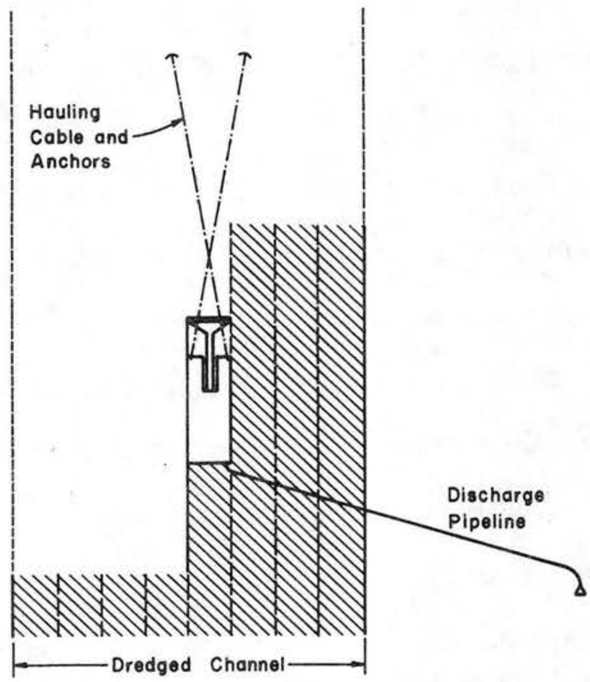


Figure 7-6 Operation of a Dustpan Dredge (after NEDECO, 1965).

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 (a direct application of the momentum principle--Equation 2.13). 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 rapidl for longer lines or for an increase in lift beyond the few feet requir 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 capacitie ranging from 1000 to 1800 cubic yards per hour. Natural conditions suc 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 disturbances 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.

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.

7.2.3 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. Normally, 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 normally 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 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.

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 up 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 (from continuity--Equation 2.6) and thus, an increase in shoaling. It was estimated 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 more than compensated for by the increased efficiency of the dredging. For example the production curve for a typical cutterhead dredge (Figure 7-7) 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).

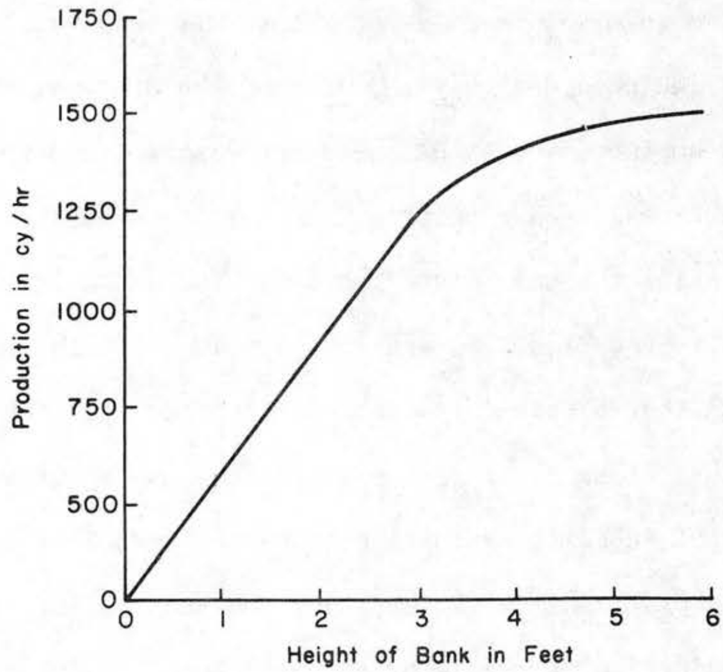


Figure 7-7 Production Curve for 30-inch Hydraulic Cutterhead Dredge Pumping through 2000-3000 Feet of Discharge Pipeline (after Hyde and Beeman, 1963).

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.

7.3 The Hydraulics of Dredged Cuts

7.3.1 Analysis

Dredging of a channel through a crossing or shoal should be considered successful if the dredged cut meets two criteria. First, the cut should achieve the required navigable depth with a minimum of excavation, and second, the cut should not require redredging 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 (see Section 3.4.2). 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.

The second criteria, that the cut should not require redredging during the navigation season, is also influenced by alignment, depth, and width of cut. Ideally, a successful dredged cut would not require redredging 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 cu 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. Here, 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, t cut does not show a tendency to fill and may even scour under favorabl 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 stabili can be based on the Chezy equation (Section 2.1.4.3) for uniform or nearly uniform flow in open channels:

$$V = CR^{\frac{1}{2}} S_f^{\frac{1}{2}} \qquad (7.$$

where:

C = Chezy's discharge coefficient

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

$S_f = \frac{h_L}{L}$ = energy gradient = $\frac{\text{Head Loss}}{\text{Length of Channel}}$

V = average flow velocity

Substituting the definition of shear stress (Section 2.1.4.4):

$$\tau_o = \gamma R S_f \qquad (7.$$

where:

τ_o = bed shear stress

γ = unit weight of fluid

yields:

$$\tau_o = \frac{\gamma V^2}{C^2} \tag{7.3}$$

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 2.75) 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, Equation (7.3) can be rewritten:

$$\tau_o = \frac{\gamma Q^2}{C^2 W^2 D^2} \tag{7.4}$$

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 (2.78), transport is reduced by 54 percent (Lagasse, 1975).

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 2.3.3.3) was guided by Einstein's bed-load function, and his graphical relations (Figure 2-31) incorporate both field and laboratory data. Accordingly, they are more representative of the physical processes involved than a simple shear stress analysis.

Colby's curves can be replotted into a form that is well adapted an analysis of the effect of width or depth change on transport capaci For a given bed material size and water temperature, the curves of Figure 2-31 are plotted as lines of equal sediment discharge per foot of width, q_t, on a velocity-depth field, as sketched in Figure 7-8.

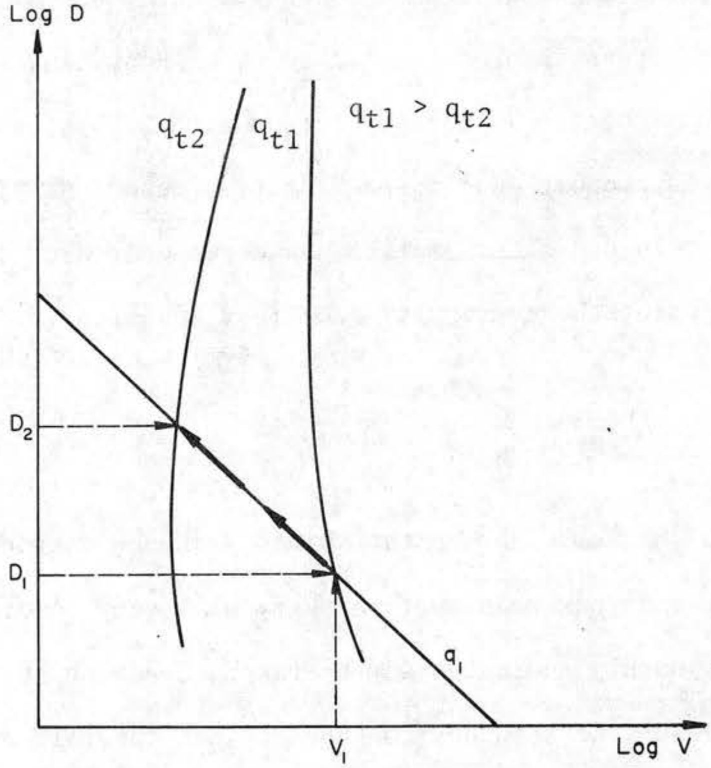


Figure 7-8 Colby Transport Relations Replotted for Width and Depth Change Analysis (after Nordin, 1971).

Lines with a slope of minus one are lines of equal unit water discharg q. The initial velocity and depth, V_1 and D_1, establish the initial unit sediment discharge, q_{t1}, and unit water discharge, q_1. For an increase in depth to D_2 at constant width, q remains constant. Thu q_{t2} can be established by following the q_1 line to an intersection with D_2 (Figure 7-8). 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_{t1} at 25 tons per day per foot of width. An increase in depth to 12 feet yields a q_{t2} 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 the width and depth of the dredged cut 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 (Section 2.1.4.3). Here, the Chezy equation is used:

$$V = C(RS_f)^{\frac{1}{2}} \tag{7.5}$$

where:

C = the Chezy discharge coefficient

R = hydraulic radius

S_f = energy gradient

<u>Assuming that slope and resistance are constant</u>, 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 7-9 is a definitic

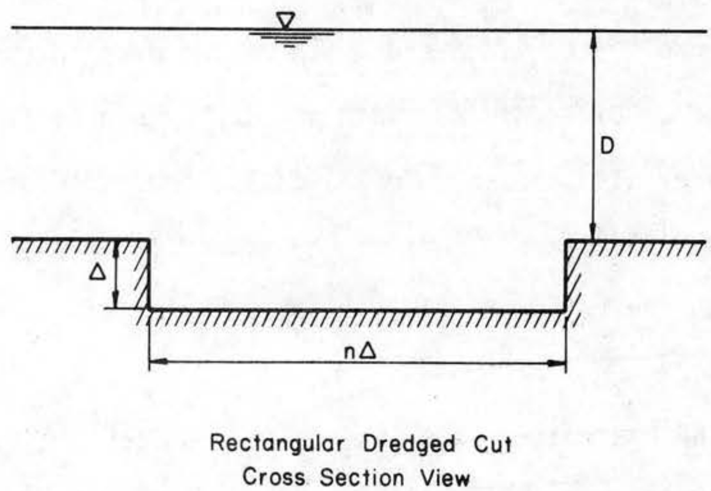


Figure 7-9 Definition Sketch--Rectangular Dredged Cut.

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 (7.5) to th undisturbed condition:

$$V_o = C(R_o S_f)^{\frac{1}{2}} = C[\frac{(n\Delta)(D)}{(n\Delta)} S_f]^{\frac{1}{2}} \qquad (7.$$

After dredging, the flow velocity on the dredged cut becomes:

$$V_1 = C(R_1 S_f)^{\frac{1}{2}} = C[\frac{(n\Delta)(\Delta+D)}{(n\Delta+2\Delta)} S_f]^{\frac{1}{2}} \qquad (7.$$

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

$$\frac{D}{n\Delta} = \frac{(\Delta+D)}{(n\Delta+2\Delta)} \qquad (7.$$

and:

$$n\Delta = 2D \tag{7.9}$$

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 7-1 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 7-1 represents

Table 7-1 Values of Hydraulic Radius for Combinations of n and Δ* (Lagasse, 1975).

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 $n\Delta = 32$

*Undisturbed Depth, (D), = 16 feet

the equilibrium condition of $R_1 = R_o = 16$, or $n\Delta = 32$. It should be noted that since:

$$R_1 = \frac{n(D+\Delta)}{n+2} \tag{7.10}$$

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 7-10 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 indica 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 widt of 20 feet (n = 10) would be unstable.

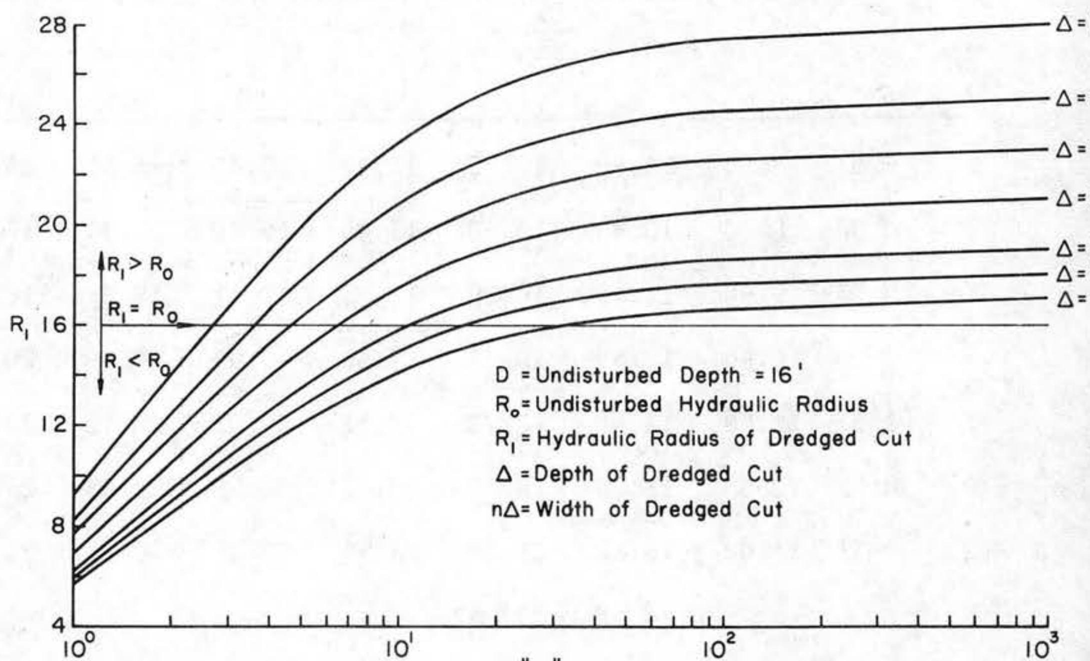


Figure 7-10 Rectangular Dredged Cut Design Curve--Steady, Uniform Flow (after Lagasse, 1975).

Cutterhead and dustpan dredges develop a dredged cut in a distinctly different manner. An analysis based on the hydraulic radiu of the cut indicates that each mode of operation has certain advantage 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 7-10 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 affect 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.

There is some indication, then, that under optimum conditions a dredged cut can be stable. However, the complexity of alluvial channel flow (Equation 2.68) 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. The effect of alignment on stability and the influence of unsteady flow resulting from a normal seasonal hydrograph must also be considered. Valuable 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.

7.3.2 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 7-11a) the

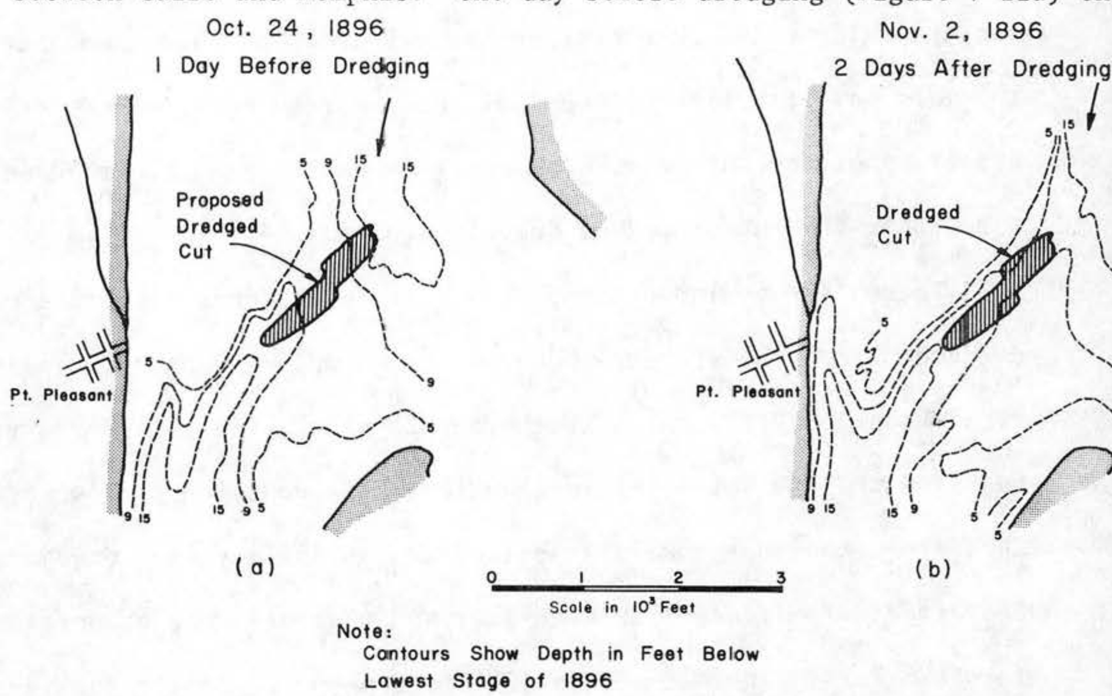


Figure 7-11 Dredging of the Lower Point Pleasant Bar, Lower Mississippi River (after Ockerson, 1898).

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 7-11b, 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 7-12a) an

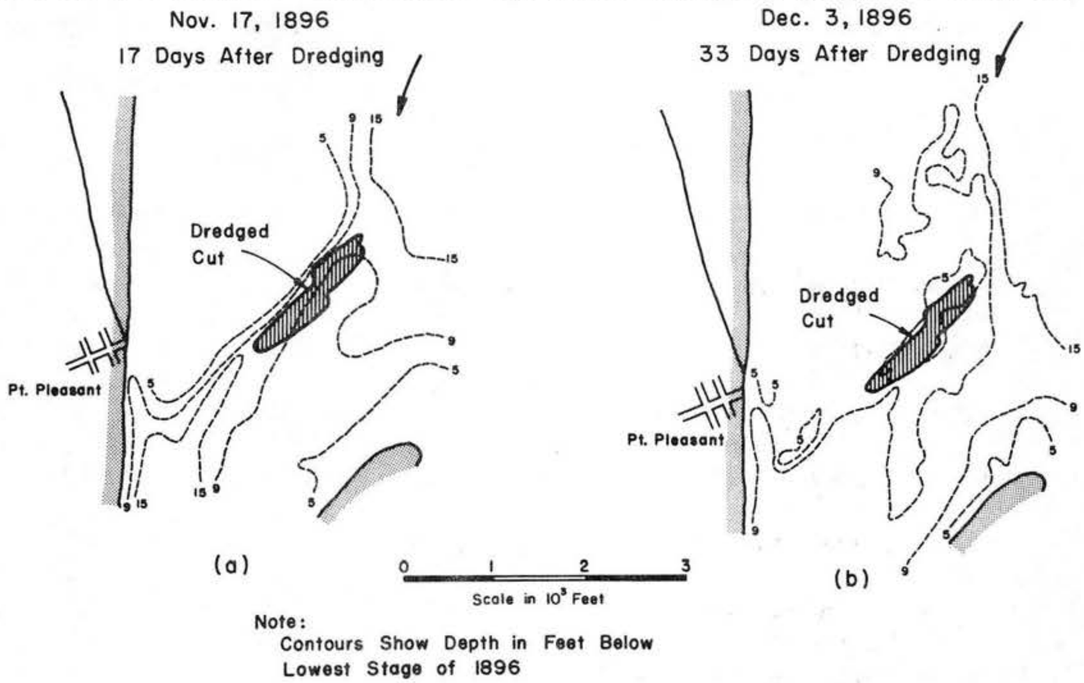


Figure 7-12 Dredging of the Lower Point Pleasant Bar, Lower Mississippi River (after Ockerson, 1898).

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 7-12b, 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 th 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 Islan 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 fal 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 yard occurred. Experimental dredging in 1931 in this vicinity, together wi 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 chara terized by frequent fluctuations. In the light of previous analysis t 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.

7.3.3 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 (Section 2.4.3) 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 7-13), 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

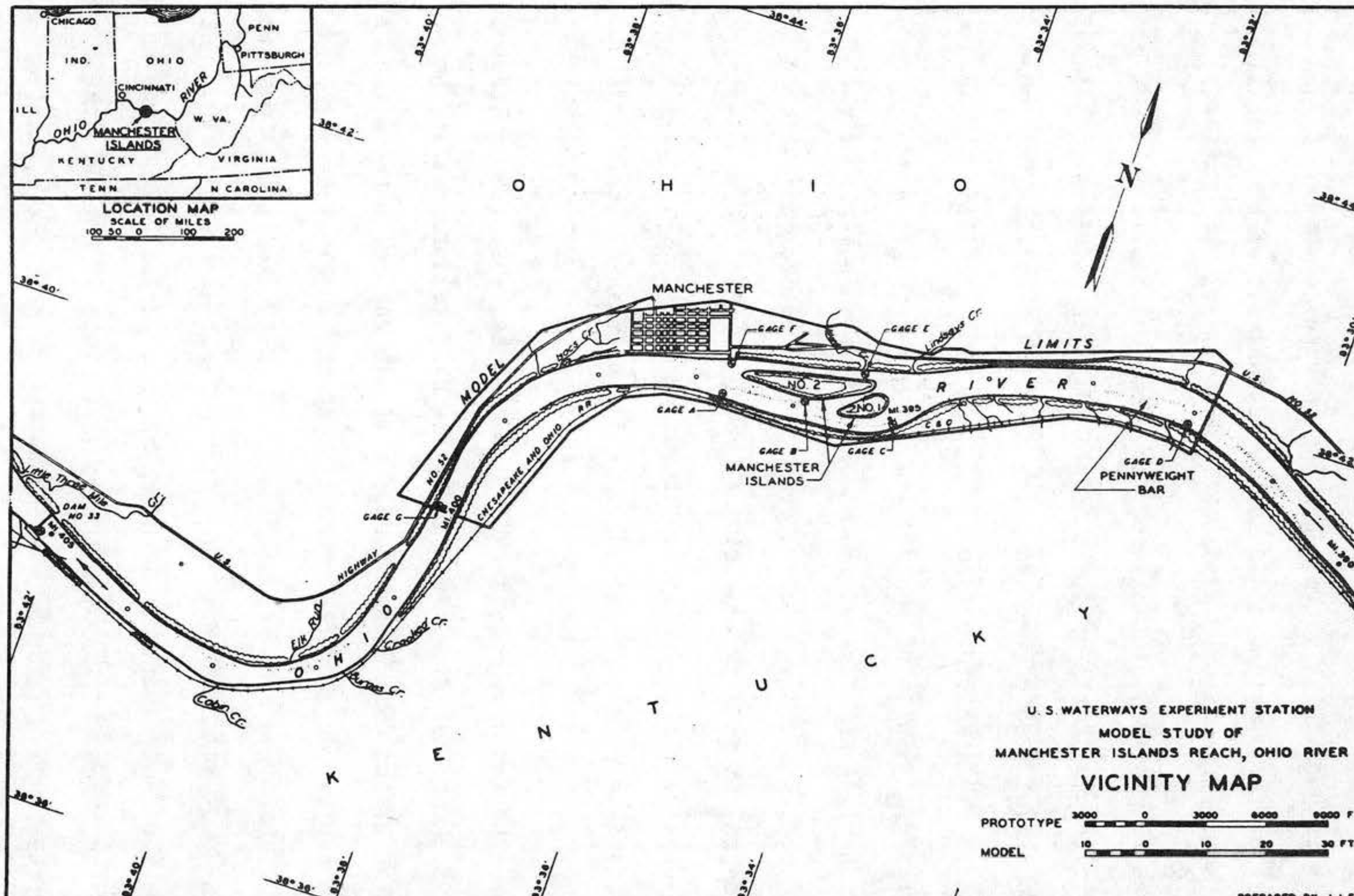


Figure 7-13 Manchester Island Reach and Model Limits (after Corps of Engineers, Waterways Experiment Station, 1941)

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 7-13).

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 (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 7-13). 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 grave 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 involved the dredging of the existing Kentucky channel to proje dimensions and the closing of the middle channel by means of a dike of dredged material. Plan B 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 channe between the two islands. Plan C envisioned dredging the navigation channel through the Ohio channel and partially closing the Kentucky channel with a dike of dredged material.

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 thos 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 7-14). As with the prototype, significant shoaling occurred i the model at the upper end of the dredged cut. During the 1935 dredgi season two primary disposal sites were used for dredged material: the

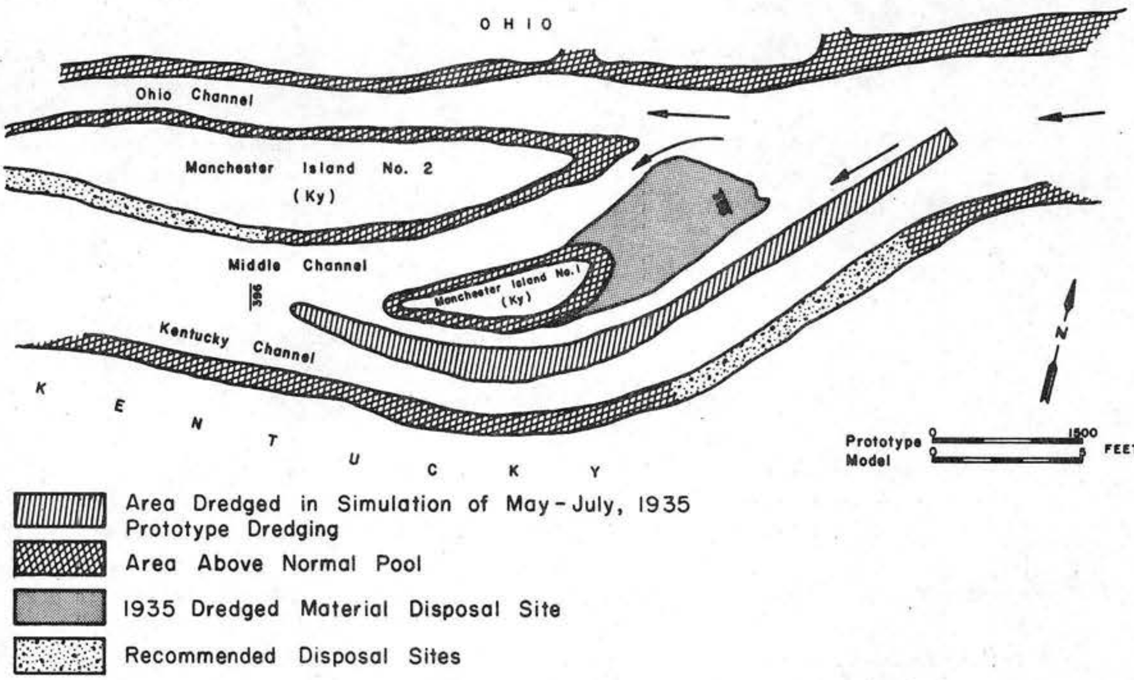


Figure 7-14 Existing Conditions--Manchester Islands Model Study (after Corps of Engineers, Waterways Experiment Station, 1941).

head of Island No. 1 and the middle channel between the two islands (Figure 7-14). 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 3.5.3).

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. During high flow, material was scoured from the point bar in the next bend upstream (Figure 7-13--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 indicat that the thread of maximum velocity cut across the head of the dredgec 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 Islanc 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 coincic 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. The consequences are also apparent. The unstable conditions in the dredged channel above and below Island No. 1 can be relat to strong high-stage crosscurrents that impinge on the cut at each location. Both locations are also downstream from a high stage sedime source area. Pennyweight bar contributes material that deposits in th head of the dredged cut, and dredged material disposal sites above and between the two islands contribute material that form the bar below Island No. 1, impacting the dredged channel there.

The results of this model study highlight 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 (through the Kentucky and middle channe respectively) were oriented properly for low-stage flows, high-stage flows produced crosscurrents that in both cases resulted in shoaling in the head of the cut. Under Plan C (dredged cut through the Ohio channel) both low and high-stage flows were forced into the navigation channel and shoaling at the head of the channel did not occur. However, the channel downstream of the cut did experience shoaling. 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 the primary conclusion from this study was that continued dredging apparently offered the most practical solution to the shoaling problem in the Manchester Islands reach. Although the more permanent approach of installation of training dikes in the model appeared to solve the shoaling problem, secondary effects such as exceptionally high current velocities in the navigation channel 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 was investigated in two additional mode studies by the Waterways Experiment Station. Both studies relate to t development of a navigation channel on the Arkansas River. In the fir 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 th 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 dredgi required for the initial cut and maintenance dredging (Franco and McKellar, 1973). This conclusion supports earlier discussion of the consequences of overdredging.

7.3.4 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 3.4.2). 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 (Section 7.3.2). 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. 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). It was generally accepted at the time that crossings or bars were built up on falling river stages which follow high and intermediate flows and were scoured at lower stages. 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 7-2. Also shown are the trends of the

Table 7-2 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)

stage (rising, falling, or stationary) and the number of dredges mobilized in the District to maintain the navigation channel during th period. This information is summarized graphically in Figure 7-15.

The peak of the hydrograph in Figure 7-15 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 en of August and then relatively stationary water levels until November. The combined action of the river and the dredging necessary to maintai a channel of project dimensions during this period lowered the average

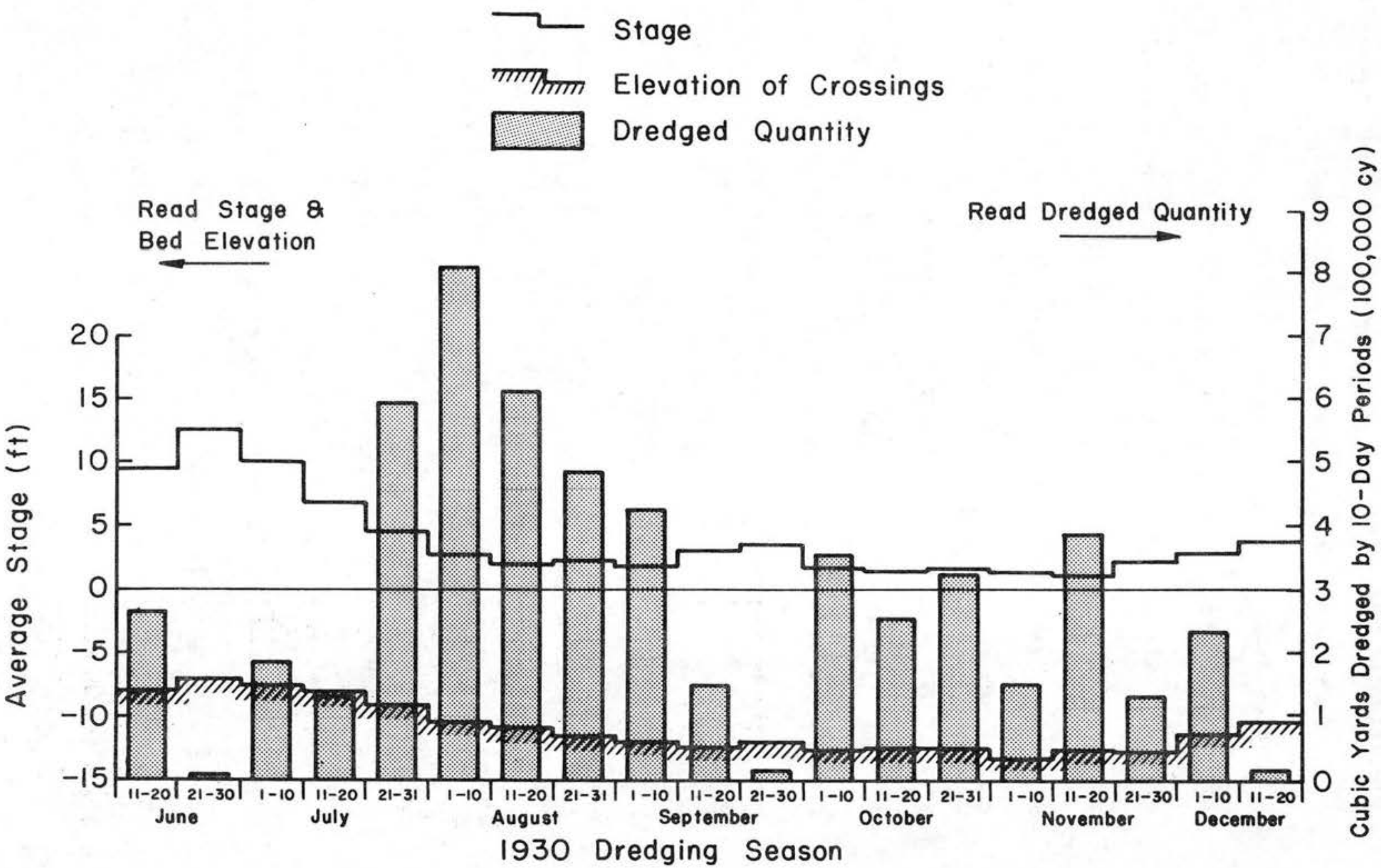


Figure 7-15 Relation of Stage to Depth on Crossings and Dredging Requirements.

elevation on the crossings by 6.2 feet. The rapid increase in dredgir 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-foc 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 durir 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 requir ment. During the 30-day period, 21 October to 30 November 1931, almos 1,475,000 cubic yards of dredging was required to maintain the navigat 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 hig and low water can be obtained from soundings on 27 crossings made by t Memphis District at the beginning of the 1929 and 1931 low-water seaso 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 relativel long period of high water while 1930-31 represents a prolonged low-wat 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 scour on the falling stage was well below normal. As a result, the average depth on a crossing was only 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 feet below the Memphis mean low water, an average increase in 7.5 feet in depth (compared to 1929) 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. 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 Upper 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).

7.4 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.

7.4.1 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, a would also induce extensive degradation of the stream bed downstream from the navigation dams (recall the qualitative analysis of response to clear water release below a dam--Figure 5-4). Project design included taking advantage of this degradation by increasing the spacin and reducing the number of navigation dams from that required to match the natural river profile. The navigation channel just downstream fro each lock and dam was to be developed initially by dredging and contra tion 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; Hallmark 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.

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 stabilizaiton 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.

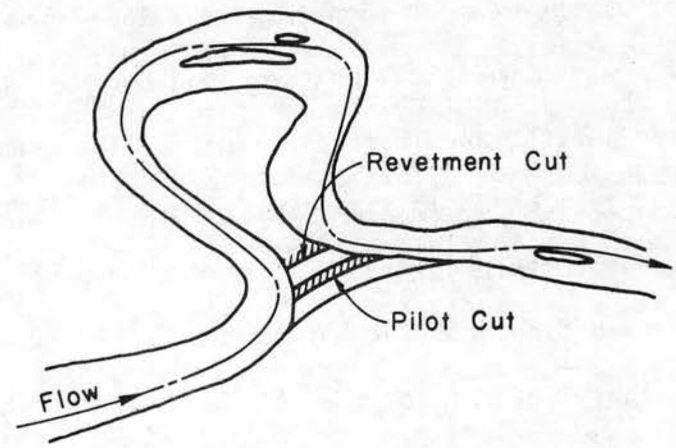
A fourth stabilization technique, contraction works, was considered secondary, and Carey suggested the possibility "that massive correcti dredging can do directly, promptly, and with certainty what contractic works do indirectly, belatedly and with uncertainty." The concept of comprehensive river stabilization 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, bu points out that the greatest possibilities for comprehensive river stabilization exist today on the unimproved alluvial rivers in the underdeveloped areas of the world.

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.

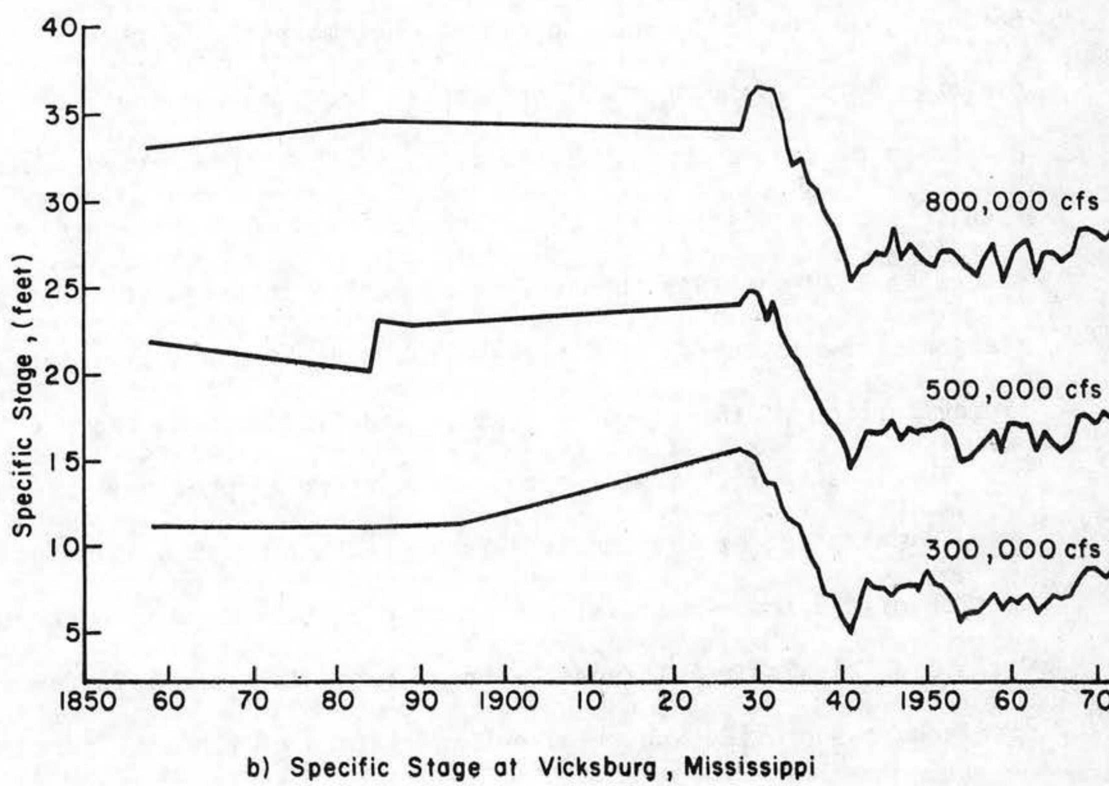
Among the dredging operations that support a program of comprehensive river stabilization, the dredging of a pilot channel for cutoff of a meander loop clearly casts dredging in the role of a morphologic agent. While cutoffs occur naturally in a meandering stream, dredging can be used to greatly accelerate 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 7-16a) 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 slack water 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.

The impact of the cutoff program on river morphology is revealed by an examination of the change in stage with time at a selected cross section of the Lower Mississippi. A plot of this "specific stage" at Vicksburg, Mississippi (Figure 7-16b) between 1858 and 1973 for a range of discharges provides an excellent indication of the long term impacts of man's activity on the Lower Mississippi. Here the direct impact of the cutoff program in reducing stages for all discharges is apparent in



a) Natural or Dredged Cutoff



b) Specific Stage at Vicksburg, Mississippi

Figure 7-16 Cutoff of a Meander Loop by Dredging and Impact of Cutoff Program as Revealed by a Plot of Specific Stage between 1858 and 1973.

the trend of decreasing stages between 1929 and 1941. The adjustment of the river to cutoffs can be seen in the general trend of gradually rising stages subsequent to 1941.

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 (recall the qualitative analysis of the response to channel straightening--Figure 5-5). Besides upsetting the slope of a river, cutoffs can 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.

7.4.2 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 stabilit Coarser materials are removed from the river only after due considerat has been given to possible adverse effects. Where gravel mining is permitted, improvement of the reach of river is the primary considera- tion, and the obtaining of gravel only a secondary benefit. The suppl of gravel is currently being exhausted in many reaches of the Lower Mississippi River. Where the mining of gravel exceeds the natural rat 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 stabi- lizing channel patterns and bed forms. This coarser fraction, particu larly 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 eros 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).

Because armoring is a scour related phenomena, it is apparent tha 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 (For a detailed discussion of the armoring process see Livesey, 1963). Dredging for gravel is generally concentrated at these locations where the material is readily availabl

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 inch 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 on the Lower Mississippi 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 45,000 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:

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 c 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 and the upper end of bars or island 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 Distri changed significantly between 1968 and 1971. The median grain diamete 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 bee 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 percen 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 river bed and thus limit the depth of scour. Armor on the upstre nose of a point bar will resist formation of a chute channel and the development of a divided reach. Through examination of gravel dredgin 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 7-17). Divided flow conditions at Victoria Bend steadily

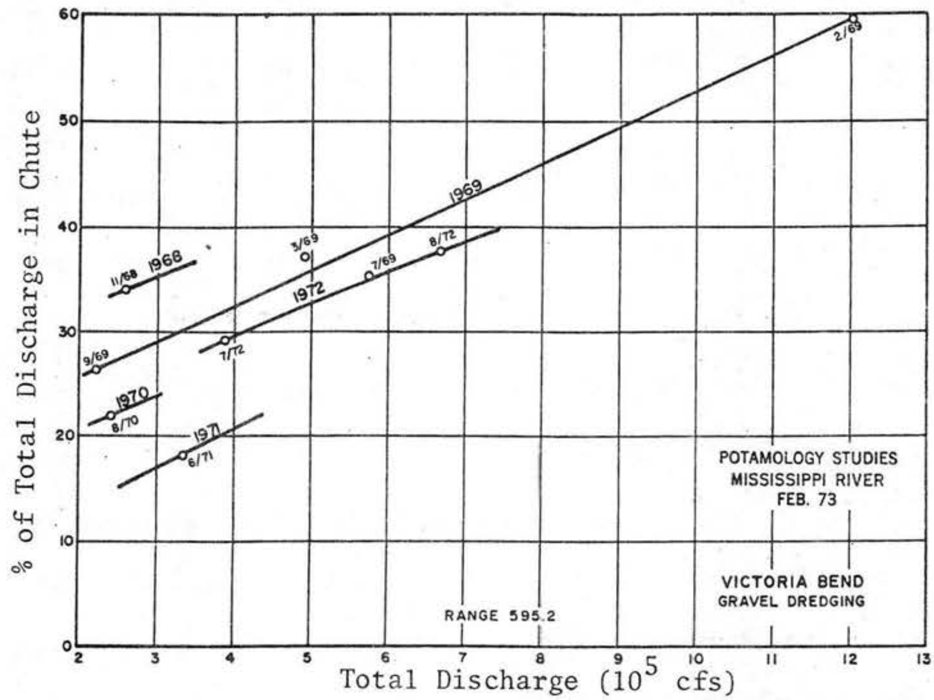


Figure 7-17 Victoria Bend Gravel Dredging, 1971-1973 (after Winkley and Harris, 1973).

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 (Figures 3-8 and 3-9), the impact of gravel mining can be expected to be 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 quadrup 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 Mississip 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.

7.4.3 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 an 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 dredgin operations for navigation channel maintenance. Dredging and open wate 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. It is significant that field experience with navigation channel maintenance problems generally leads to the conclusion that dredging alone is usually unsuccessful in obtaining and assuring full project depth. As a result, the combined use of dredging and contraction dikes to maintain project dimensions is usually recommended (Lagasse, 1975).

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 7-18b. It was concluded that the increased cross section could

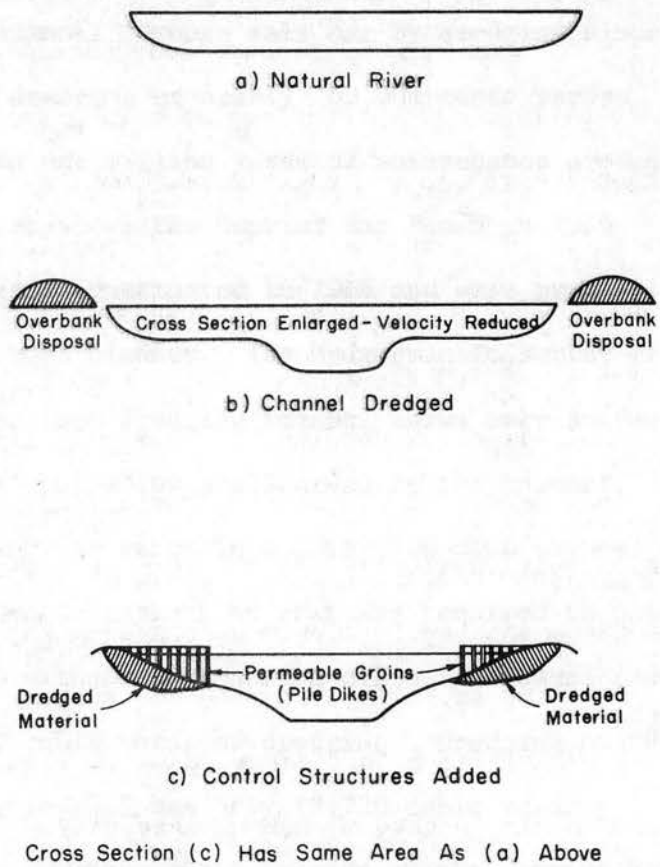


Figure 7-18 Schematic Cross Sections--Channel Dredging Concepts.

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 7-18c 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 7-18a and 7-18c. 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. 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 7-19a shows soundings taken on the bar in 1909, before annual

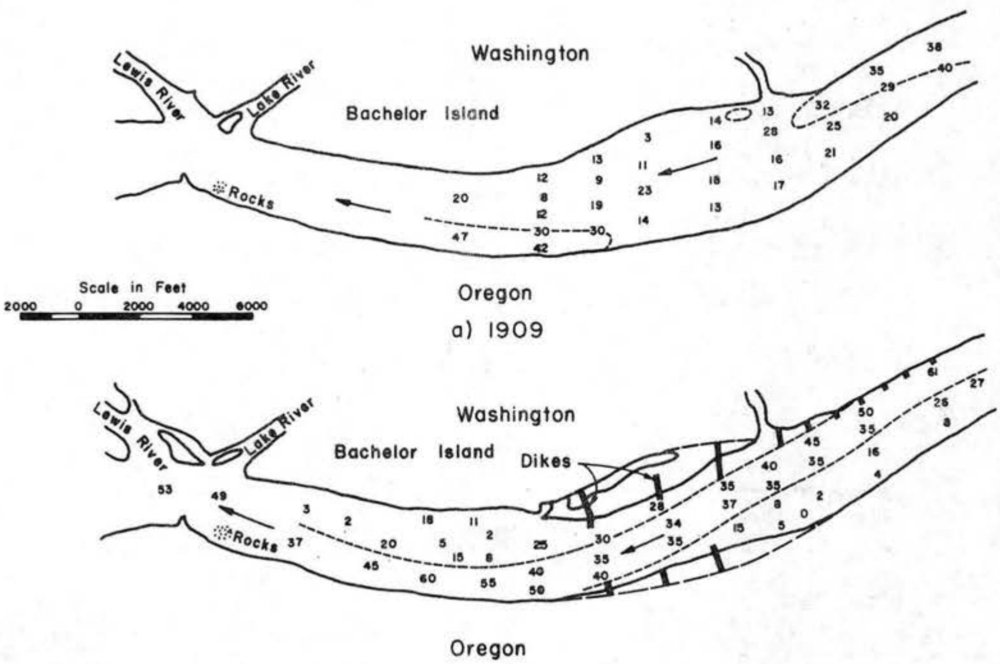


Figure 7-19 Henrici Bar, Columbia River (River Mile 91), 1909-1959 (after Hickson, 1965).

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 requirec

Figure 7-19b 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 7-19b 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, 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 7-20. The 1909 natural width of 4000 feet

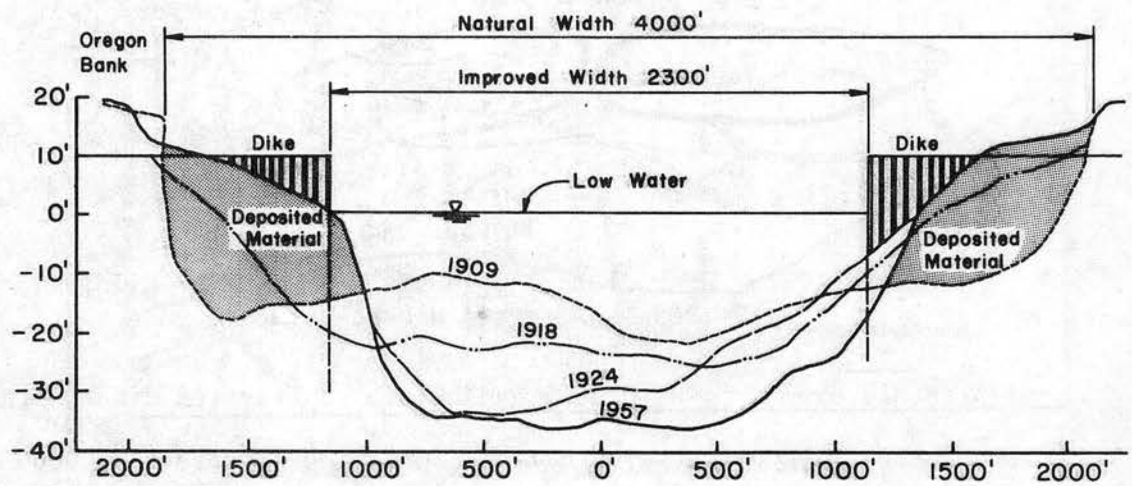


Figure 7-20 Cross Sections at Henrici Bar, Columbia River (River Mile 91),1909-1959 (after Bubenik, 1963).

was decreased to 2300 feet in 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. The Portland District's engineering concept for the 35-foot by 500-foot navigation channel included 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 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 (Section 5.3) provides an opportunity to investigate in detail the impact of the lateral redistribution of sediments on channel shape.

The combined effect of dikes and dredging can be evaluated by comparing change in river cross section in this reach 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 7-3. 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

Table 7-3 Cross-Sectional Data at Locations with Dikes and Dredging

Location	Date Year	Stage above msl	Area sq feet	Top Width feet	Av. Depth ($\frac{\text{Area}}{\text{Top Width}}$)	$\frac{\text{Width}}{\text{Depth}}$
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%

sections is striking. On the average, width to depth ratios decreased by 53 percent between 1891 and 1940.

The Crider Island section of RM 280 in Pool 24 is typical of these cross sections. During the early 1880's (Figure 7-21) Dikes 3

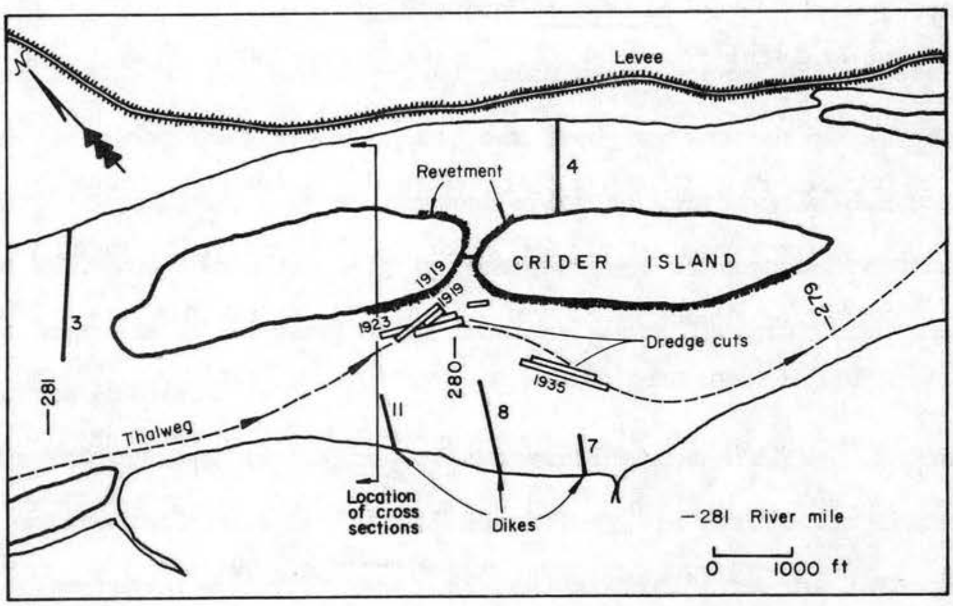


Figure 7-21 Map of Crider Island.

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 7-22. The closure of the chute channel

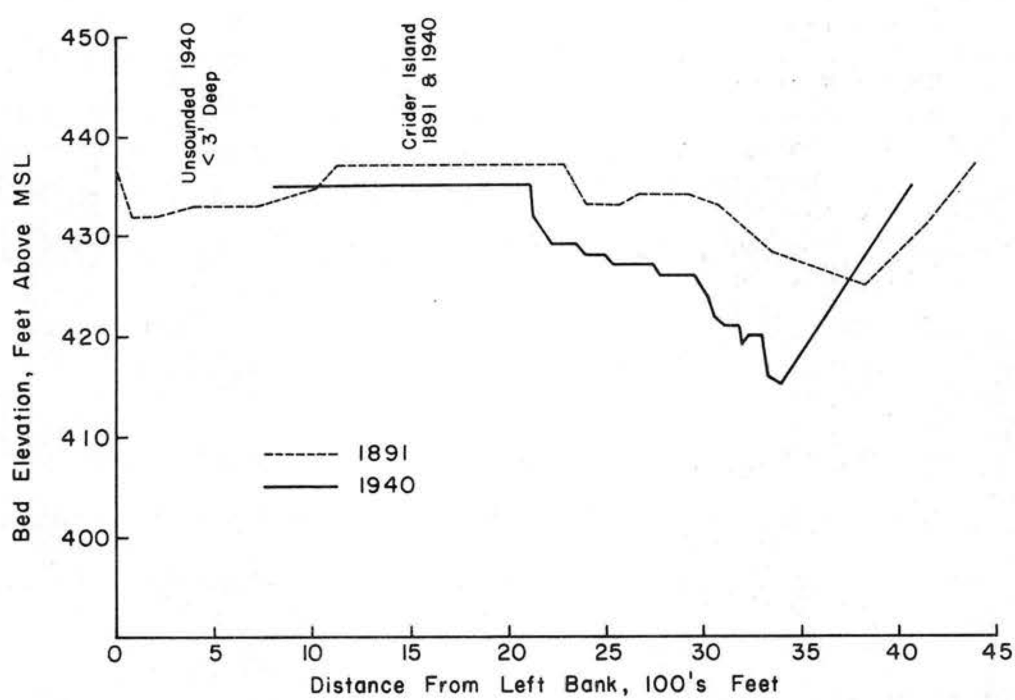


Figure 7-22 Crider Island Cross Section (RM 280).

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 7-4. The Middleton Island reach has no main channel dikes, but dredgi

Table 7-4 Cross-Sectional Data--Middleton and Angle Islands

Location	Date year	Stage above msl	Area sq feet	Top Width feet	Av. Depth ($\frac{\text{Area}}{\text{Top Width}}$)	$\frac{\text{Width}}{\text{Depth}}$
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

between 1923 and 1936 removed 57,000 cubic yards from the main channel The width to depth ratio in this reach <u>increased</u> 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 channe 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.

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.

Many researchers have shown that 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 (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 (Lagasse, 1975).

7.5 Dredging and the Nine-Foot Channel Project

A comparison of geomorphic data developed in Chapter 5 with dike construction and dredging locations in the Pool 24, 25, and 26 reach has indicated that contraction efforts and dredging at the same locati can significantly alter channel shape, and thus sediment transport capacity. In this section geomorphic data is combined with an analysi of the dredging records for the lower pools of the Upper Mississippi navigation system 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 Mississipp and under conditions of open river regulation as on the Middle Mississippi.

7.5.1 Analysis of the Dredging Records

For the Pool 24, 25, and 26 reach on the Upper Mississippi, dredgi data has been compiled by the Rock Island and St. Louis Districts sinc in 1906. Dredging volumes by year for each pool are summarized in Figures 7-23, 7-24 and 7-25, respectively. Annual dredging quantities

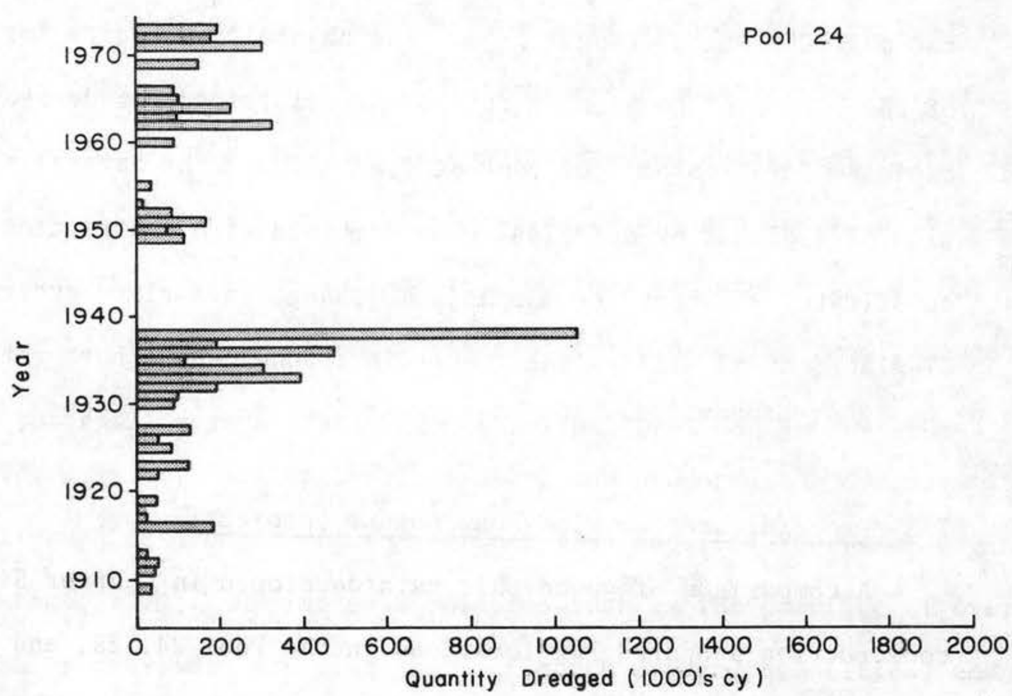


Figure 7-23 Dredged Volume by Year in Pool 24.

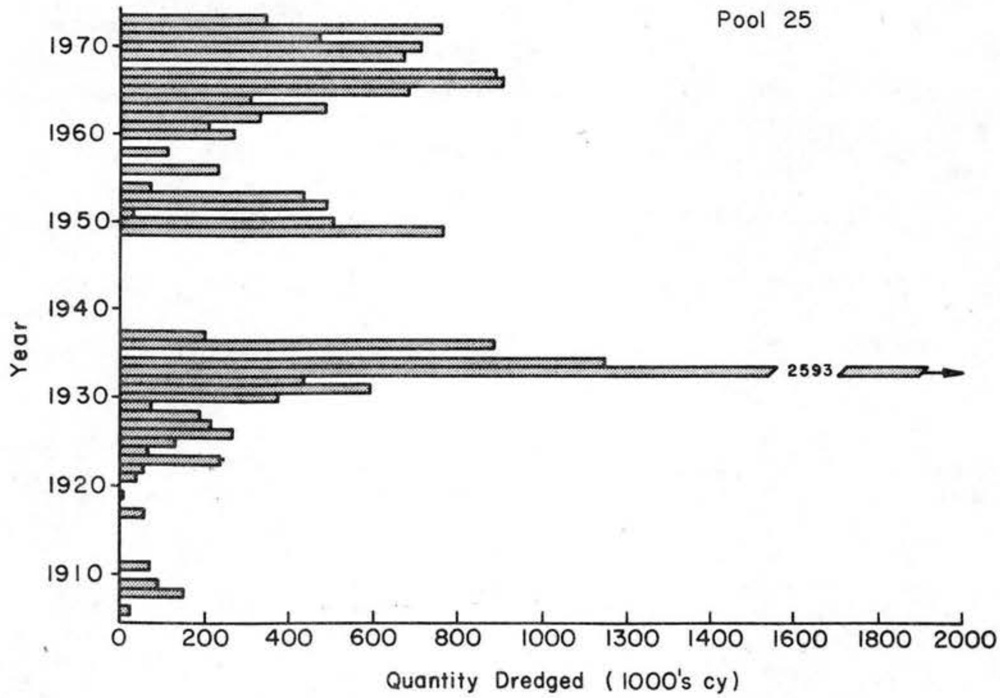


Figure 7-24 Dredged Volume by Year in Pool 25

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.

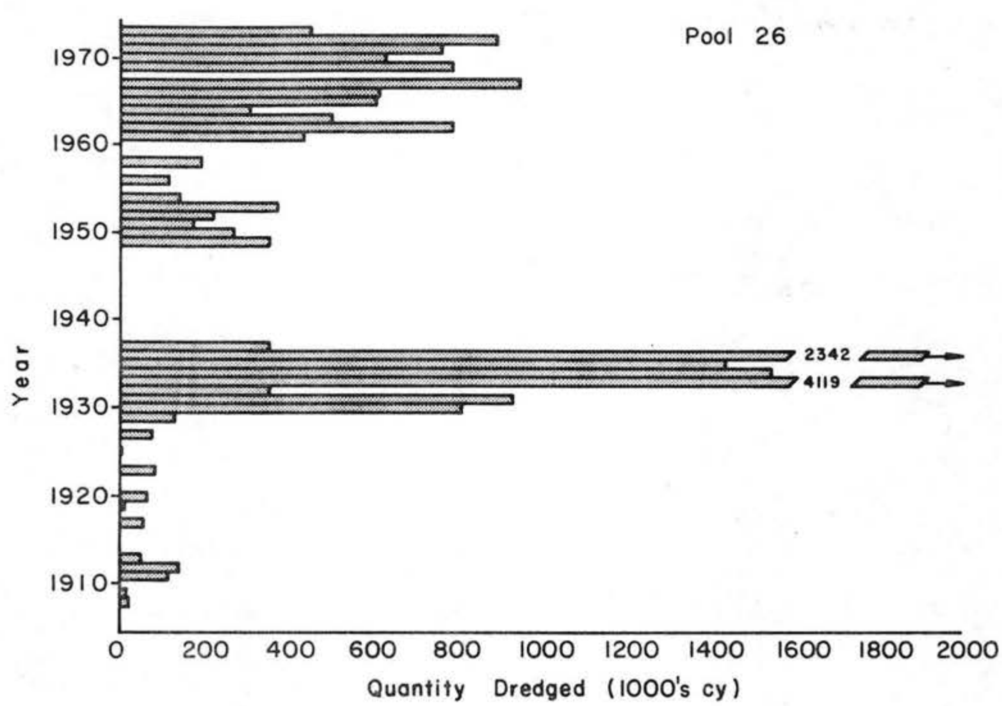


Figure 7-25 Dredged Volume by Year in Pool 26.

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 7-5. While dredging quantities remained essentially constant in Pool 26, average annual dredged volur increased by 66 percent in Pool 25 and decreased by 22 percent in Pool 24.

Table 7-5 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	-

Establishing a direct cause and effect relationship between dred volumes in a reach of river and geomorphic or hydraulic parameters is difficult. Not only do the number of engineering variables influenci dredging requirements cloud the issue, but such factors as funding an dredge plant availability also influence the quantity of material dredged in a given year. For example, dredged quantities in the St. Paul District by the Dredge Thompson began a sharp decrease in 1955. However, this decrease was not related to a geomorphic trend, but rather to assumption of Rock Island District dredging requirements by the Thompson. These additional dredging requirements resulted in post poning some St. Paul District dredging and produced the decrease in the record of dredged volumes between 1955 and 1964 (Corps of Enginee St. Paul, 1974). Operational policies such as the practice of over-depth 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 in a given study area.

In Pools 24, 25, and 26 the large dredging volumes of the 1933-1938 period (Figures 7-23, 7-24, 7-25) 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 5-28) was below the gage zero in six out of seven years. Although dredging studies by the Memphis District (Section 7.3.4) 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 bedload 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 5-28) has exceeded 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 crosscurrents and resulting fill of the cut. The studies by the Memphis District in the early 1930's relate stage and dredging requirements (Figure 7-15) 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 7.3.1) and are apparent in the results of the model study of Lock and Dam 8 on th Arkansas River (Section 7.4.1). Here, it was concluded that increasin 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 the initial cut and long-term maintenance dredging quantities were considered.

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 (1963) 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 an increase of dredged volume per unit water discharge.

In contrast, 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 (Section 5.3.4). 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 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 7-26. 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

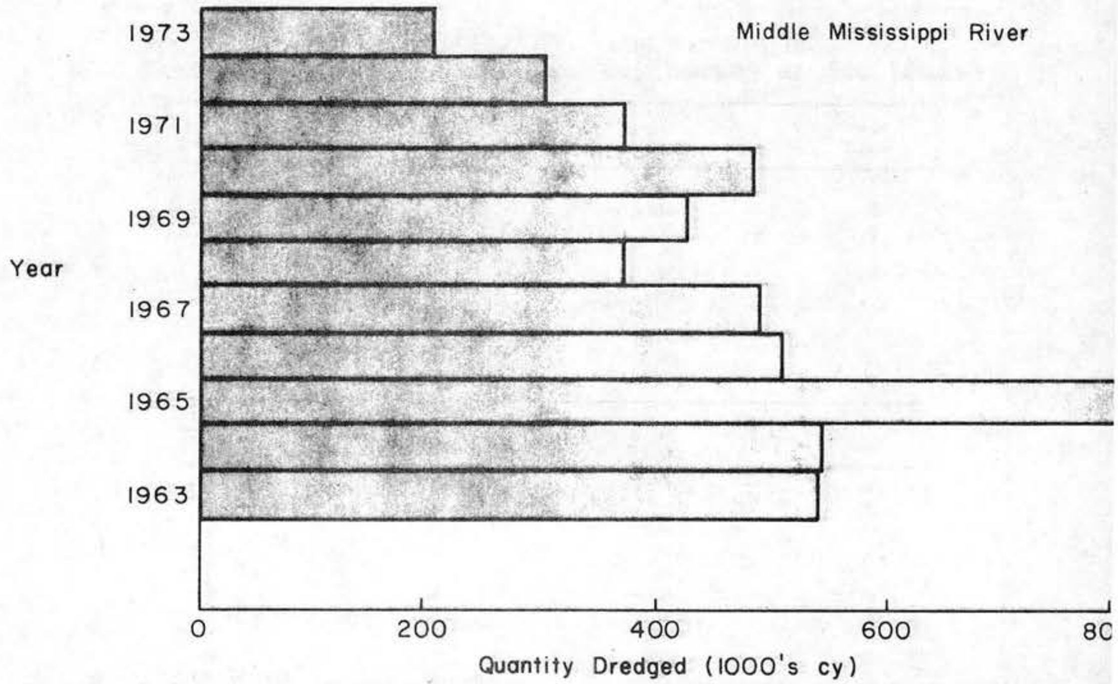


Figure 7-26 Dredged Volume by Year in the Middle Mississippi River.

channel project. 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 (called the prototype reach) to a 1200-foot width, the current 1500-fo width contraction plan was adopted. The result has been generally decreasing dredging requirements on the Middle Mississippi.

Using the prototype reach from River Miles 140 to 154 as an examp average width in the reach was reduced from 4800 feet in 1889 to 1800 feet in 1966 and the river bed was lowered by an average of 8.0 feet. Between 1967 and 1969 the prototype reach was narrowed to 1200 feet and the river bed dropped an additional 3.0 feet. The effec of this man induced geomorphic change on dredging requirements can be seen in Figure 7-27. Between 1963 and 1974 dredged quantities increas

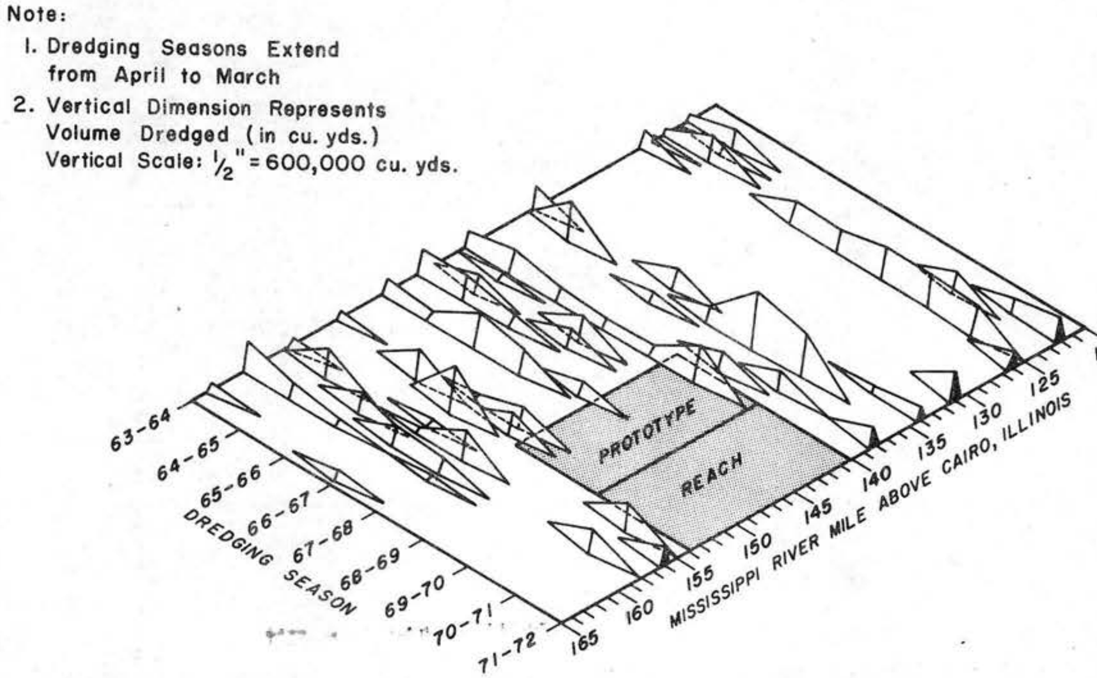


Figure 7-27 Isometric Drawing of Dredging History, RM 165 to RM 120, Middle Mississippi River (after Degenhardt, 1973).

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 7-27 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 (Figure 6-20). At St. Louis 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 7-6 and Figure 7-28. Table 7-6 shows a reversal of the stage-discharge order in that floods of recent years

Table 7-6 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)

with relatively low discharges have produced stages that are among the highest of record (see Section 6.4.3.2 for a detailed discussion). Th flood, for example, ranked only 10th in discharge but produced the hig stage of record (43.3 feet) at St. Louis. Figure 7-28 shows the trend of changing stage for a given discharge on the natural river and on th developed river at St. Louis. Using this figure, it is estimated that 1844 record discharge of 1,300,000 cfs which produced a stage of 41.3 at St. Louis would now pass St. Louis at approximately 52.0-foot stage

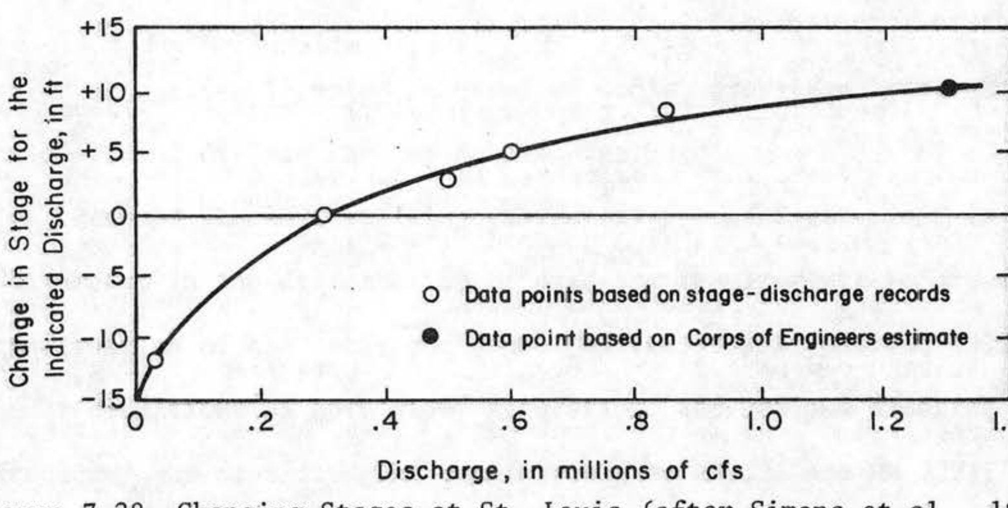


Figure 7-28 Changing Stages at St. Louis (after Simons et al., 1974).

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 river development 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 7.3.3).

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 (Table 5-5) is an indicator of sediment moving through the Pool 24, 25, and 26 reach. 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 7-5), 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 reac but about 57 percent of the contact load. In view of the impact of t lateral redistribution of contact load on channel morphology and considering the importance of contact load to the determination of river form and character, interrupting the down-system movement of more than 50 percent of the contact load in a 100-mile reach of river is significant.

7.5.2 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 Again, using the Pool 24, 25, 26 reach, dredging frequencies by location from River Miles 200 to 300 are shown in Figure 7-29 for the 1918-1938 pre-lock and dam period, and for the 1949-1973 post-lock ar dam period of the dredging record. Also indicated on Figure 7-29 are the locations of the four locks and dams in the reach, 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 7-29 is the difference in the distribution of dredging requirements before and a construction of the locks and dams. In the 21-year period, 1918-193 under conditions of open river regulation, the maximum dredging frequency was nine dredging events (between RM 221 and RM 222), and

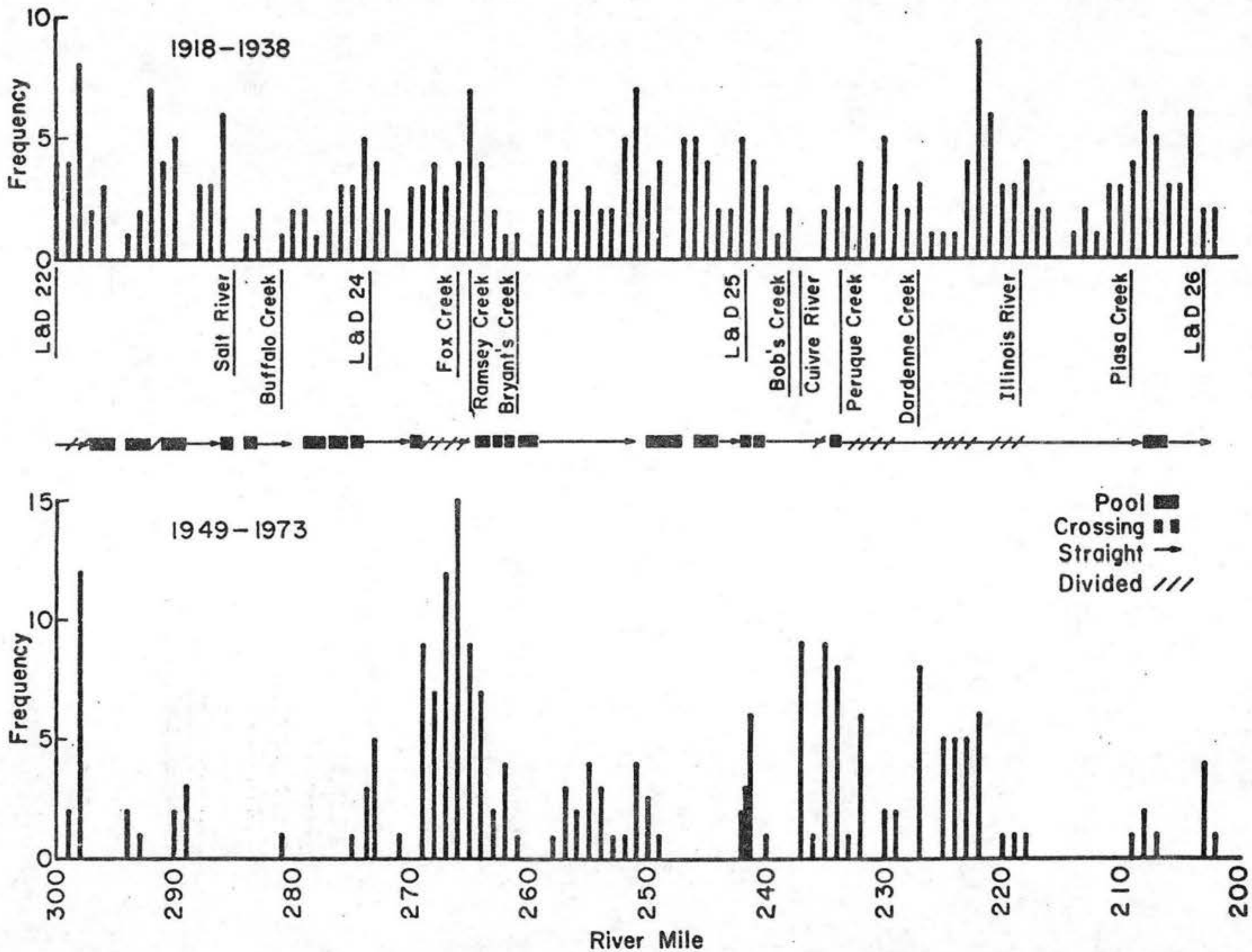


Figure 7-29 Dredging Frequency by Location in Pools 24, 25, and 26, Upper Mississippi River.

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 location: 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 this reach radically altered the distribution of dredging requirements.

While the dredging frequency plot of Figure 7-29 tends to highligl dredging trouble spots, it also includes locations that have required only infrequent dredging. In addition, Figure 7-29 does not consider dredged volume which can be as strong an indicator of a dredging troub spot as frequency. Accordingly, Figure 7-30 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 i 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 require 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 7-30. Tributary, lock and dam, and pool control point locations as well as the general morphologic character of the river are shown. The

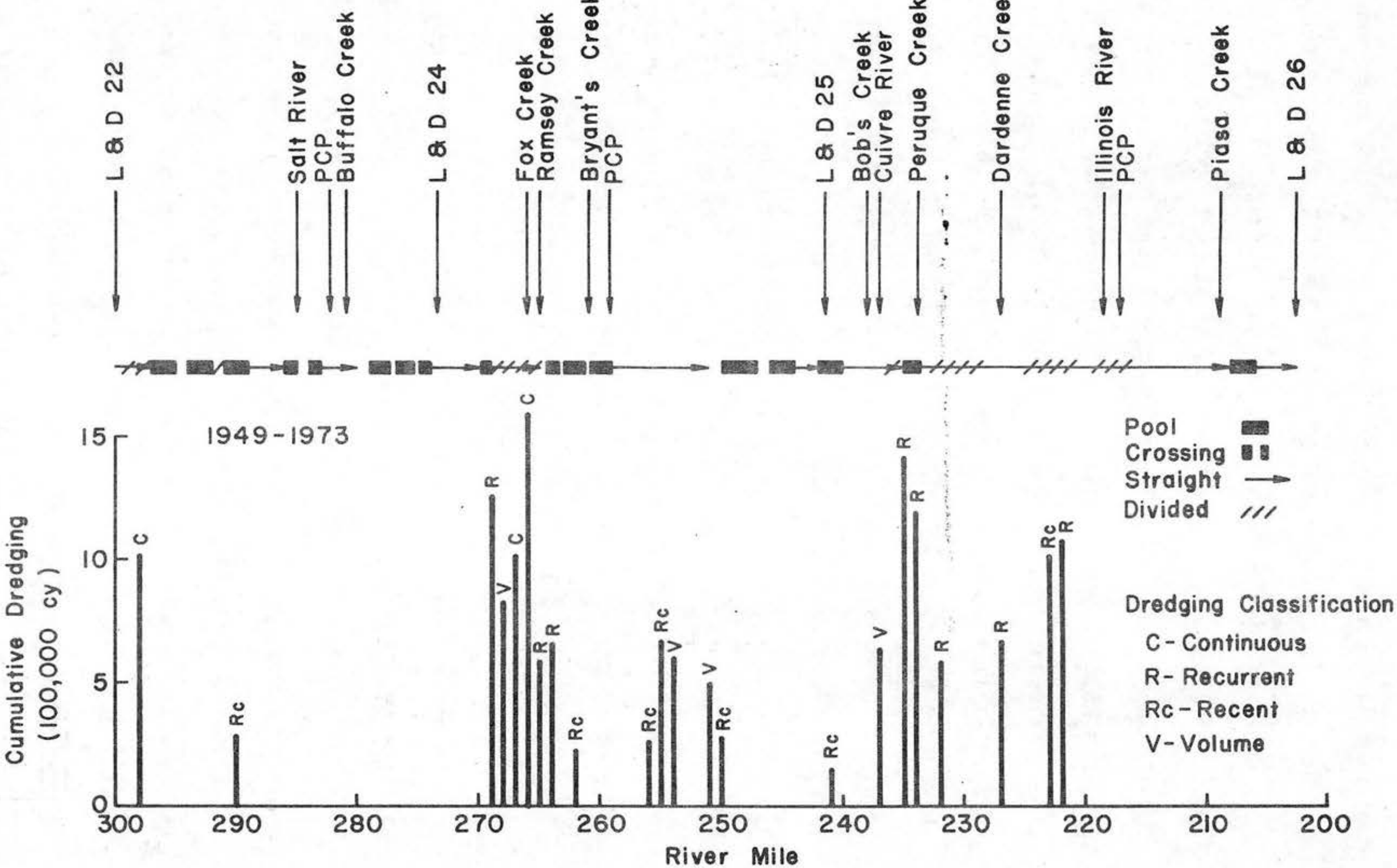


Figure 7-30 Location of Continuous, Recurrent, Recent, and Volume Dredging Problem Areas in Pools 24, 25, and 26, Upper Mississippi River.

relationship between geomorphic factors and dredging requirements is investigated using Figures 7-29 and 7-30 as the primary references.

Contrary to what might be expected, dredging in this reach of the Upper Mississippi 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 (Figures 7-29 and 7-30). The combined influe of four smaller tributaries, Bob's, Peruque, and Dardenne Creeks, and the Cuivre River, between River Miles 228 and 238 has contributed to a shift in the Mississippi from the western bankline toward the eastern bankline (Section 5.3.2.1). Moreover, Rubey (1952) attributed the for tion of Peruque, Dardenne, and Cuivre Islands to these tributaries. Although the reach bracketed by these tributaries contains locations c recurrent, recent, and volume dredging, the relatively low water and sediment discharge of these tributaries dictates 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 requirement 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 (Figure 4-1). It has been estimated that the Chippewa is responsible for about 20 percent of all maintenance dredgi along the Mississippi River between Pool 4 and Pool 10, 148 miles downstream (Corps of Engineers, St. Paul, 1974).

Both Figures 7-29 and 7-30 reveal a strong correlation between lock and dam locations and dredging requirements. A location requirin continuous dredging exists just downstream from Lock and Dam 22 at River Mile 298. The particularly troublesome River Miles 260 to 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 7-29 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 (Figures 5-2 and 5-4). The longitudinal profiles through Lock and Dam 24 (Figure 5-20) for the 1930-1970 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 5 (Section 5.3.1.2). Pools 24, 25, and 26 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 (PCP) for Pools 24, 25, and 26 are located at Louisiana (RM 282.9), Mosier Landing (RM 260.3), and Grafton (RM 218.0), respectively, and are shown on Figure 7-30. Dredging trouble spots in

Figure 7-30 tend to be located above the pool control point in each pool, where the river is flowing under essentially open river conditic In Pool 24, 100 percent of the continuous, recurrent, recent, and volu dredging occurs between the primary control point at Louisiana and Pool 22, upstream. In Pool 25, 73 percent of the trouble spot dredgir 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 thi portion of the pool apparently provide depths in excess of the require navigable depth on most crossings and, thus, reduce dredging requirements.

Field experience has established that most dredging operations ar conducted to improve depths on the shallow crossings or to increase channel width adjacent to point bars in bends. While the scale of Figure 7-30 does not permit a detailed correlation of dredging require ments 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. Th unstable nature of straight reaches is described in Chapter 3 (Section 3.2.1). 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 downstrea In relating the location of dredging trouble spots to straight reaches between River Miles 200 and 300, Figure 7-30 reveals that of the three locations requiring continuous dredging, all three are located in straight reaches. Of the eight locations of recurrent dredging, six a 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 7-29 and 7-30. Reaches were classed as divided only if they contained large alluvial islands with relatively open chute channels, such as Gilbert Island (RM 298), Carroll-Slim Islands (RM 229-233), and Bolters-Iowa Islands (RM 222-226).

The morphology of divided reaches was outlined in detail in Chapter 3. Rubey's data (Figure 3-21) 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 3-22) point to significant sedimentation problems at the entrance to and exit from a divided reach. 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 7.3.3). Maintenance dredging problems can be anticipated, then, when a navigation channel passes through a divided reach.

The dredging data from the Pool 24, 25, and 26 reach supports thi 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 River Miles 264 and 269 can be related to the division of a straight reach into multiple channels by the Carroll-Slim Islands complex. Thi 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 7-29 and 7-30 reveal a close correlation of dredging trouble spots in the Pool 24, 25, and 26 reach 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 River Miles 200 and 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.

In regard to geomorphic factors, then, field experience has established that under conditions of either open river regulation or low flow regulation with navigation dams, the crossings in a meanderin 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 7-30 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.

7.6 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.

Early in 1975, the Corps of Engineers, St. Paul District initiated planning to develop a pilot program of reduced overdepth dredging for 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 7.2.3). Although these sites were dredged to less than 13 feet for reasons other than hydraul 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 navigatio season. Comments were requested concerning the sites or types of site which should be selected from the list of 1975 projected dredging locations for inclusion in the reduced overdepth dredging pilot progra 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 fro the Pool 24, 25, and 26 reach were used to devise a procedure for selecting sites for possible inclusion in the reduced overdepth dredgi 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 analys 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 stabili 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 surve 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 frequency by location provided 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 7-7. 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

Table 7-7 Selection of 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	13.78 +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

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 7-7. Although there is a degree of subjective analysis in the selection process, the factors considered in Table 7-7 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 feet immediately after dredging. 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 i 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 pilo 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 over-depth dredging program. Based on this case study, it can be concluded that an analysis of the hydraulic and geomorphic response of a river to dredging provides the river engineer with a methodology applicable to the solution of a variety of problems related to navigat channel maintenance.

7.7 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 (Section 7.4.3). 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 reach.

7.7.1 Environmental Impacts of Dredged Material Disposal

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 5.3.1.2). 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 bedforms composed of rather sterile, sandy contact-load sediments whicl 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 such physical factors as light penetration, water depths, and currents. Dredged mat deposited in peripheral areas of the channel can affect both benthic a 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 extent of 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 fis In many cases, these habitat conditions continued to exist after creation of the navigation pools of the 9-foot channel project. Subsequently, the extent of aquatic habitat was greatly increased by the formation of the large navigation pools (see Figure 5-19 for example). This habitat has been continually reduced by the settling of fine sediment particles. The current practice of disposing dredged materia in the dike fields and peripheral channel areas can accelerate this process of filling.

River marshes form a vital ecological transition zone between the open water aquatic habitat and terrestrial habitats. Marshes draw fro 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 impact of dredged material disposal in a marsh by direct elimination of extremely valuable habitat can be severe. 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 water fowl 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 chapter. 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 critic 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 o 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 sourc for wild fowl. These plants protect some important food organisms fro predatory fish, thus, reserving more of the river's production for terrestrial species. Sandy beaches are also important to the reproduc tion 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 alluvia islands and along the channel periphery provide a firm anchor for disposed material. Wind and water transport of dredged material is signi icantly 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 strands of willow and cottonwood is smothered by the disposed material, both willow and cottonwood are quite toleran 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.

Although, the effect of dredging on water quality in the riverine environment is a primary environmental concern, this problem has not been investigated in sufficient detail to permit the formulation of definite conclusions or guidelines. 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 th summer of 1973 to determine the immediate effects of dredging activiti on water quality in Pool 8 of the Upper Mississippi River lock and dam system. Sampling stations were established in the immediate vicinity Crosby Slough and Crosby Island (RM 690.2) where the Dredge Thompson w 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 dredgi A statistical analysis was made of the means of all parameters collect 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 di solved oxygen. It was concluded that differences in these parameters were more the result of diurnal changes than a function of the dredgin process. The turbidity increase was attributed directly to the dredgi 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 dredgin on water chemistry at the Crosby Island site were localized, and that significant downstream changes in the chemical parameters measured would 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 available, 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 highly polluted, the resuspension of polluted materials becomes a dominate factor in the impact of dredging on the environment.

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, recreatio 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 navigatio 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 water fowl and aquatic fur animal management.

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

7.7.2 Geomorphic and Hydraulic Consequences of Open Water Dispos

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 geomorphic and hydraulic principles of river mechanics developed in Chapters 2 and 3 constitute the necessary baseline for an examination of the consequences of open water disposal relative to the evolution o 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 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 <u>at locations where deposition would occur naturally</u>. 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 7-18). The results of this decision can be seen in the reduced dredging requirements on the Henrici Bar of the Columbia River and have been discussed relative to the morphologic effects of the lateral redistribution of sediment (Section 7.4.3). In effect, the disposal of dredged material in dike fields is in consonance with the Mobile District's general recommendation to place dredged material at locations where deposition would occur naturally, since the dike fields create a man-induced depositional environment. 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 provid 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 o alignment on the stability of a dredged cut (Section 7.3.3). Since this study resulted in recommending continued dredging as the most practical solution to the shoaling problems in the reach, the model wa 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 reac of an alluvial river. Verification tests of the model, run under existing conditions (Figure 7-14), revealed that a dredged material disposal site at the head of Island No. 1 used during the 1935 dredgin season severely impacted the morphology of the divided reach. In the prototype 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 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 (Figure 7-14). 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 th middle channel formed a bar along the left (south) bank of Island No.

which extended downstream toward the Kentucky bank. These results support 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 7-8 which should be read with reference to Figure 7-14. 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

Table 7-8 Fate of Dredged Material in a Typical Divided Reach*

Disposal Location	Stable High Stage	Stable Low Stage	Fate of Dredged Material	Location Recommended for Disposal yes	Location Recommended for Disposal 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 fo Figure 7-14

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 some instances 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 river born 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 7.7.1), the cause and effect relationship is reasonabl clear. In others, a combination of factors must be considered. The recent closing of Wyalusing Slough in Pool 10 at River Mile 627.7, whic feeds a 3600-acre backwater area, can be attributed to a combination o natural sedimentation and dredged material placed in 1968. The Weaver Bottoms-Lost Island area of Pool 5 at River Mile 745 has been affected 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 accumulat islands of dredged material, which line both sides of the navigation channel at the lower end of Weaver Bottoms, and fill or occlusion of t various sloughs which feed this extensive backwater area (Corps of Engineers, St. Paul, 1974). In the latter two cases it is difficult t assess the impact of disposal practices, as the effects of disposal ar superposed on the natural processes of accretion in side channels.

A geomorphic study of the evolution of side channels on the Middl Mississippi River (Section 3.5.2) has shown that the ultimate fate of the mature side channel is obliteration by both 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 (Figure 3-20) 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 3-19) 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. 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 undistinguishable from the floodplain (Section 6.3.3).

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 los of circulation and reduction in water quality as a result of natural man-induced deposition in feeder channels. The potential benefits fr 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 mc phology of the river system dictates that proposals to open backwater areas by dredging be examined in detail for each site. For favorably located side channels such as Picayune Chute, the procedure could hav long range beneficial effects, however, the potential for adverse imp as a result of the process also exists. Converting a closed slough i an open chute channel could greatly accelerate sedimentation, particu larly if the opening is oriented to induce the flow of sediment-laden high stage flows through the opened channel. The introduction of san main channel sediments which would settle on the organic-silt backwat bottom sediments could also produce a decrease in biologic productivi 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 Pool 5. The effects of this experiment are currently being monitored

The 1969 dredged material survey on the Upper Mississippi sugges the limited use of alluvial islands for disposal of dredged material. The recommendation was made in view of the limitations existing in sc areas for continued disposal of dredged material without detrimental effects on fish and wildlife, recreation, and navigation, and include only islands having low value for timber and wildlife. The geomorphi processes of island building (Section 3.5.2) 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 5-16) 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.

When the environmental impacts of dredged material disposal are compared with the geomorphic and hydraulic consequences of open water disposal, areas of serious conflict become apparent. Quite often, 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 7-31. It should be noted 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

ALTERNATIVES TO CURRENT DREDGING AND DISPOSAL OPERATIONS

- Erosion and Sediment Control
  - Watershed Land Treatment
  - Sediment Deposition Control Structures
  - Confined Disposal Areas
  - Shore Protection of Disposal Areas
  - Revegetation of Disposal Areas
- Placement of Dredged Material
  - Selective Placement
  - Remote Disposal
  - Central Disposal
  - Remove from Floodplain
- Dredge Operations
  - Type of Dredge
  - Cutterhead Modification
  - Reduced Overdepth and Overwidth
  - Dredge Openings into Backwater Areas
  - Increased Dredge Plant Capacity
- Uses of Dredged Material
  - Commercial
  - Recreation
  - Wildlife Habitat

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. A disposal 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 within the capabilities of existing dredge plant, is investigated in the following section.

7.7.3 Disposal of Dredged Material in the Main Channel -- An Alternative

A comparison of the environmental impacts of dredged material disposal with the geomorphic and hydraulic consequences 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 physic 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 wa a "cut and fill" approach to dredging. With this approach, the crest of a shoal or crossing which requires deepening is moved downstream in 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 alternative 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 7.2. The test was conducted over a two-month period in July and August 1971 and is estimated to have successfully removed about 50,000 cubic yards 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 problem and avoiding the impacts of wetland or shallow water disposal (Corps o Engineers, Portland District, 1973).

The concept of thalweg disposal can also be supported from a general geomorphic point of view. Although the longitudinal profile o a river can be conceptually described by a decreasing exponential relationship such as the Shulits equation (3.4) and visualized as a smooth, concave-upward curve (Section 3.2.7), the detailed longitudinal profile of a river is more complex. Detailed profiles along the Upper Mississippi (Figures 5-14 and 5-20) and the Middle Mississippi (Figure 6-21) 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 3.4, 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 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. Howeve 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 avoidi shallow water or wetland disposal with the consequent impacts on chut channels, sloughs, and backwater areas. As the main channel region o the Upper and Middle Mississippi is generally biologically rather sterile (Section 7.7.1), the direct biologic impacts of thalweg dispos would be minimal. Based on limited observations, the impacts of turbidity resulting from current dredging operations on the Mississip do not appear to be serious (Section 7.7.1). This conclusion would also pertain to the process of thalweg disposal of the relatively cle 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 influen 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 has been 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 was based on a mathematical model for water and sediment routing developed by Chen (Section 2.4.4), and was adapted by Chen and Lagasse to investigate the feasibility of thalweg disposal.

As pointed out in Section 2.4.4, the limitations of such a 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 patterns of the river morphology can be considered. To study a special reach of river in detail, either this river reach is subdivided into a larger number of segments to provide detailed results from 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 River Mile 269 to 264 reach of Pool 25 constitutes

the most serious dredging problem area in the three pools (Figure 7-
Accordingly, one crossing in this reach that has required extensive dredging and the pool immediately downstream were identified and mod by adding 18 additional sections between River Miles 269.0 and 265.0. This reduced the distance between adjacent sections to as little as 0.08 of a mile.

A dredged cut 3 feet deep and 950 feet 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 2.94). This resulted in approximately 200,000 cubic yards of dredged material which was disposed of in the downstream poc 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 7-32.

The initial reference bed level with dredged cut and disposal si is shown in Figure 7-32a (dotted line). The difference between the dotted and the solid line indicates the change in bed elevation. The dark vertical bars represent the upstream and downstream locks and dams. The discharge hydrograph, number of days after dredging, and discharge volume in thousands of cfs are shown in each figure. After a 132 day low-flow period, the bed level showed very small changes (Figure 7-32b). With the flood entering the reach (Figure 7-32c), th dredged cut was filled in rapidly and the bar at the disposal site wa moved to a downstream crossing as shown in Figure 7-32c. After one year, both crossing and pool returned to nearly the natural state. This sequence coincides with the influence of stage outlined in Section 7.3.4 and shown graphically in Figure 7-15.

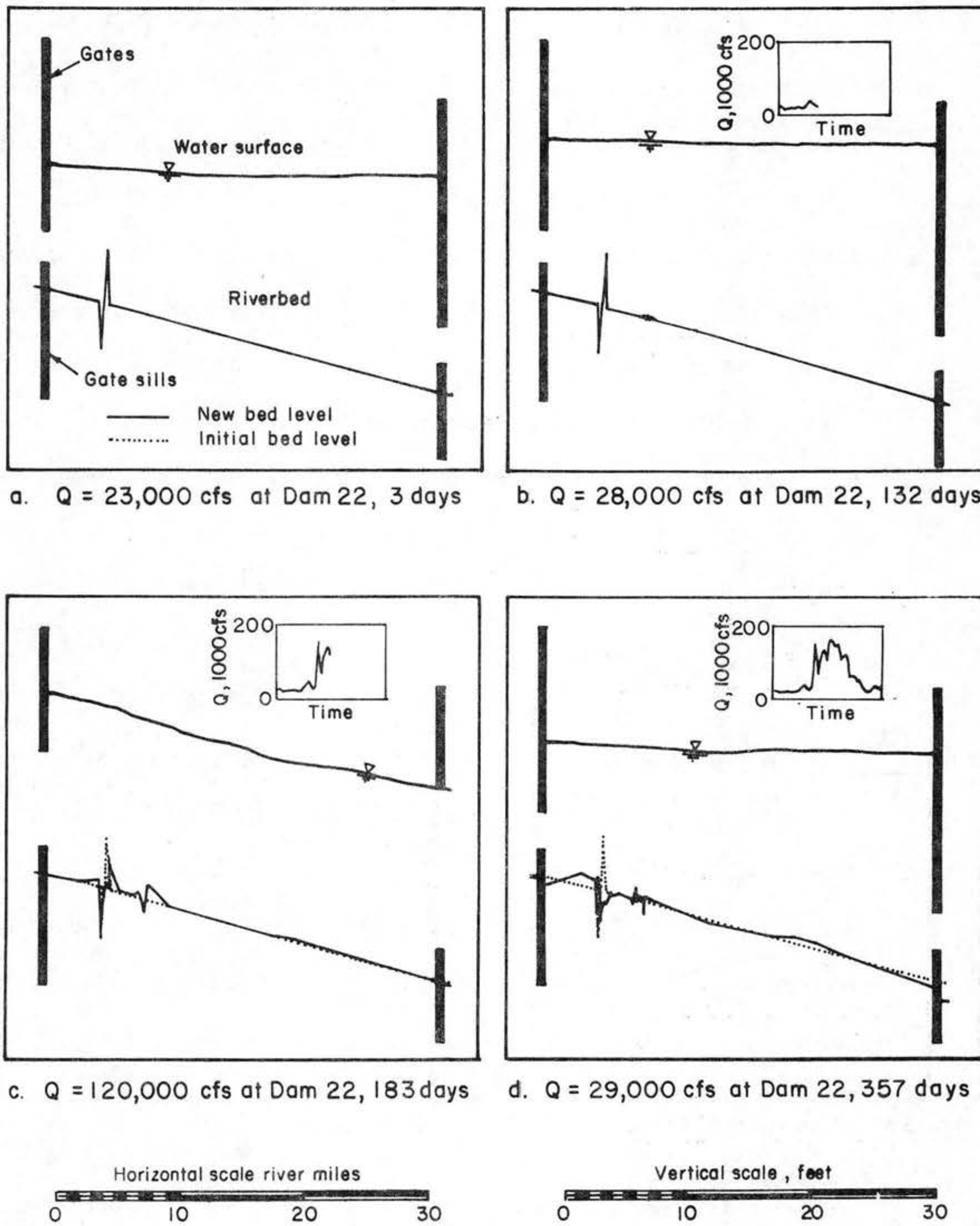


Figure 7-32 Riverbed Level Changes during the Year after Dredging and Disposing.

The sequence is illustrated in more detail in Figure 7-33 which shows the riverbed level changes with time in the crossing and the pc areas. Riverbed levels are compared with those that would occur duri the same year if no dredging had been performed. Without dredging, t crossing aggrades and the pool area degrades. Following dredging frc the crossing and disposal in the downstream pool, the dredged cut per sists for about 180 days and then begins to fill. The disposed mater in the pool also remains stable for about 180 days, but then begins t 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 throu the dredging season under the flow conditions simulated. For the location and flow conditions tested the accretion on the next crossin 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 7-34 shows the general location of the dredging and disposal operation and the impac of each alternative. With all the dredged material disposed on the floodplain (Figure 7-34b), the downstream pool scoured much more duri the year following dredging than if no dredging had been done. The sediment derived from this scour may be deposited, in part, on a down stream crossing. Disposing of half the dredged material in the downs pool (Figure 7-34c) reduced the amount of scour in the pool area and,

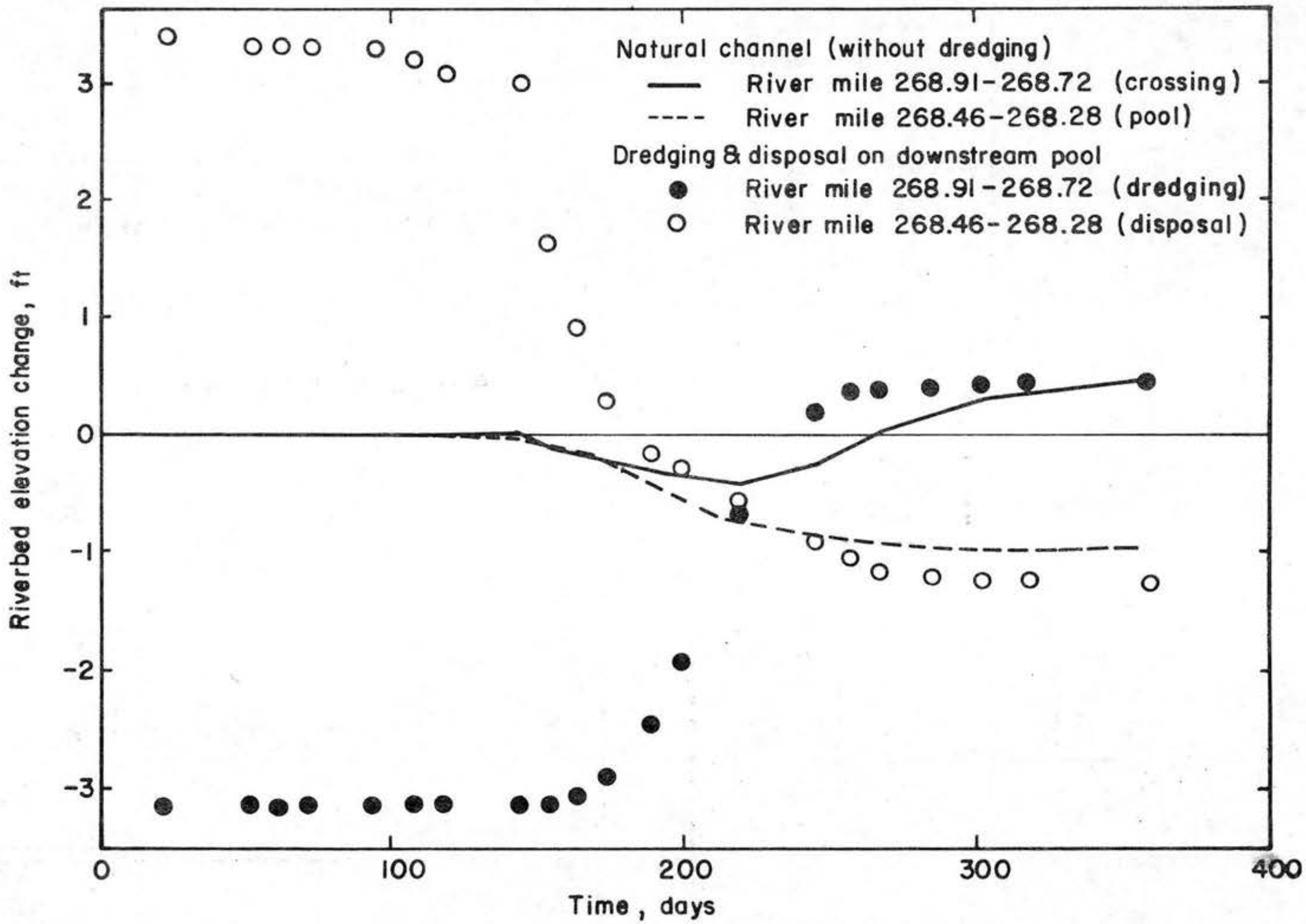
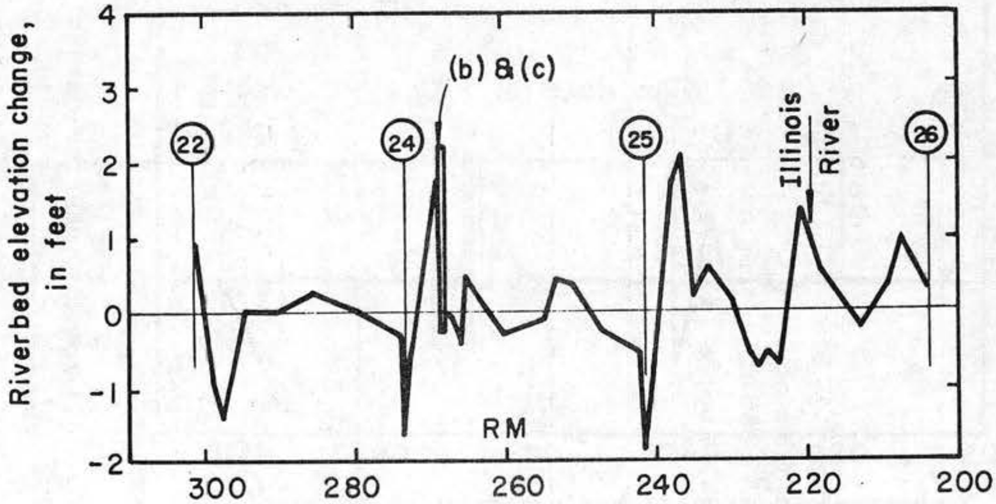
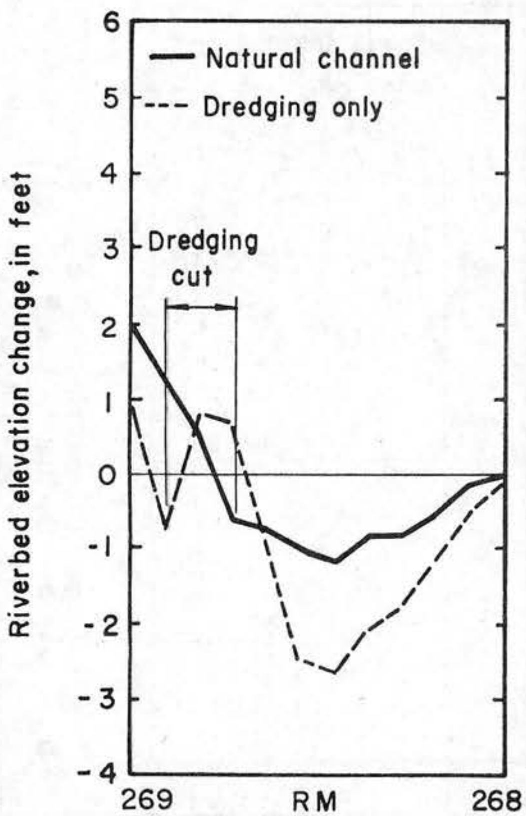


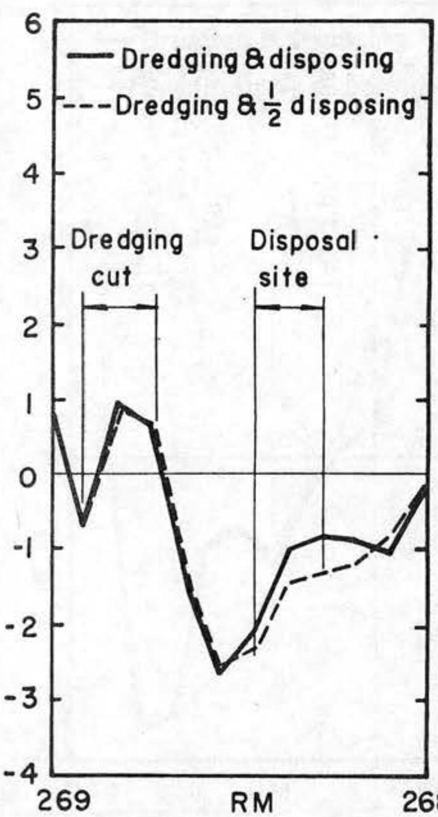
Figure 7-33 Average Riverbed Elevation Changes with Time in a Crossing and its Downstream Pool Area (2-Year Annual Hydrograph)



(a) Location of Dredging and Disposal Operations and General Riverbed Profile.



(b) Dispose All on Floodplain.

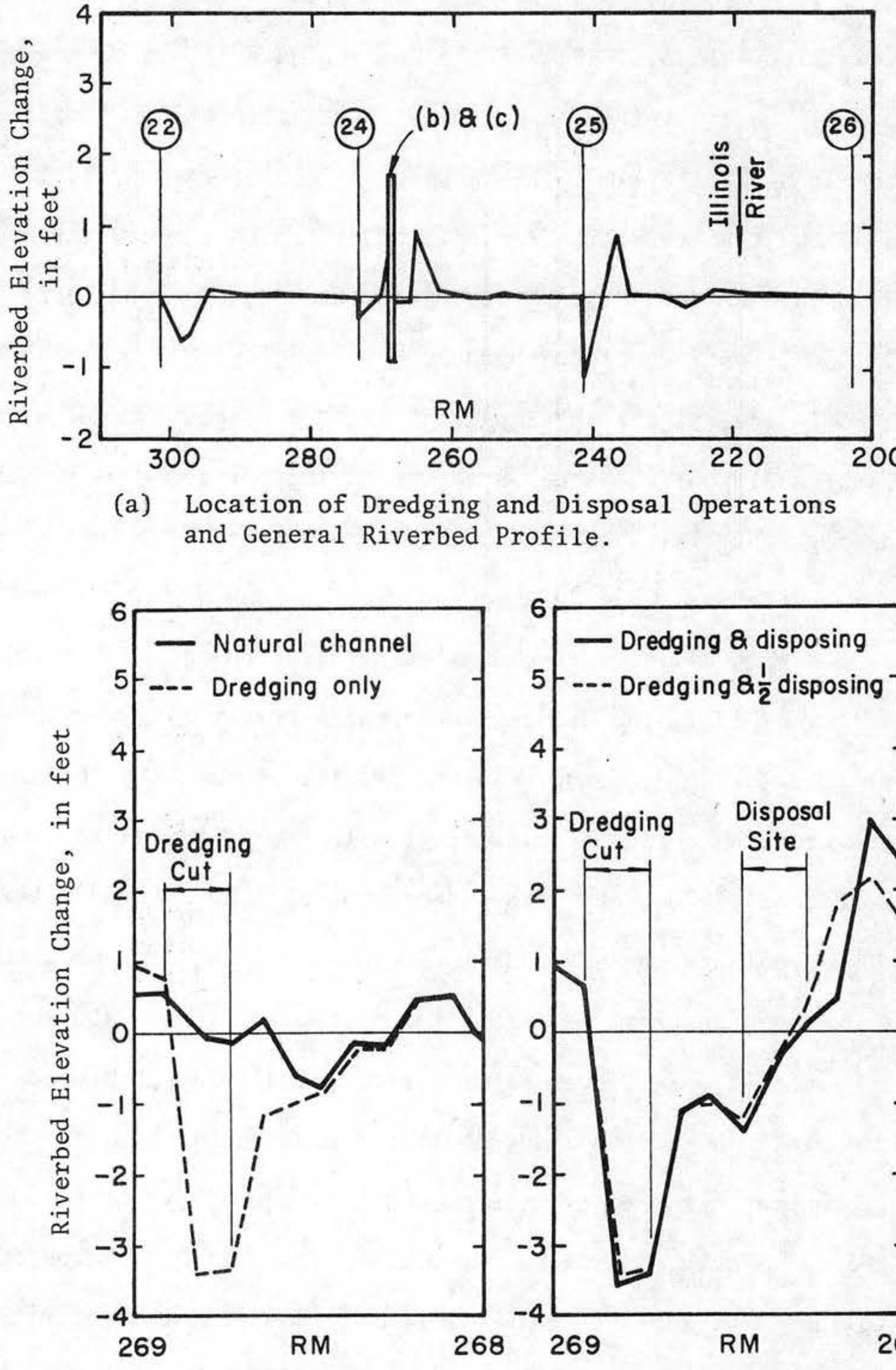


(c) Dispose 1/2 on Floodplain and 1/2 in Pool.

Figure 7-34 Average Riverbed Elevation Changes One Year after Dredging with Alternate Disposal Options (2-Year Annual Hydrograph).

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 7-34c). It should be noted that very little accretion occurs on the downstream crossing (RM 268.28 to RM 268.0) 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 one-year annual hydrograph. The results of subjecting the model to this annual hydrograph with small flow volume and reduced peak are shown in Figure 7-35. With all the dredged material disposed on the floodplain (Figure 7-35b), 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 7-35c shows accretion of 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 7-35c). 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



(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 7-35 Average Riverbed Elevation Changes One Year after Dredging with Alternate Disposal Options and a Small Annual Hydrograph.

dredging problems, and could also be true if a divided reach or unstable straight reach is located below the disposal pool.

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.

The concept of thalweg disposal may be modified for improving the existing detrimental impacts of dredged material disposal. The process involves redredging and placing the material, previously dredged and disposed on the undesirable areas, back in the main channel during high flow (Goodell,* 1975), in hope that the placed material may be flushed out

*Goodell, H., 1975, personal communication.

of the problem reach without causing unacceptable impacts. This process again involves a degree of risk and should be carefully evaluated before its applications.

The concept of thalweg disposal appears to offer 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.

CHAPTER 8

DATA NEEDS AND DATA SOURCES

8.1 Introduction

The primary purpose of this chapter is to identify data needs for the geomorphic and hydraulic analysis of river systems with particular emphasis on data requirements for general and detailed studies of the Upper Mississippi River. Although large volumes of data relative to the morphologic and hydraulic characteristics of rivers have been collected by various federal and state agencies as well as academic institutions, much of this data is not readily obtainable. Consequently, the search for data, which is a necessary preliminary step to any river system analysis, can consume a significant portion of the time and money allocated to a given study. With a view toward minimizing the investment in the data gathering effort, a listing of data sources is provided. In addition, a detailed checklist is included in this chapter to serve as both a guide for data gathering and as an outline of basic considerations for the analysis of the impact of historical and proposed development activities on the river environment.

8.2 Data Needs

The data needed for the geomorphic, hydraulic, and environmental analysis of river systems can be categorized as follows: (1) topographic and hydrographic data, (2) geologic data, (3) hydrologic data, (4) hydraulic data, (5) environmental data, and (6) climatologic data.

8.2.1 Topographic and Hydrographic Data

Topographic and hydrographic data can be obtained from topographic survey, hydrographic survey and aerial and other photographs. These data are needed to locate bluff lines; determine extent of floodplain and position of the river channel; and to determine channel geometry to include river meanders, sandbars, braided or straight sections, islands and chute channels, cross-sectional shape of the river channel, and its longitudinal profile.

An area map is useful to identify and correlate the location of the development project and all streams, structures and encroachments involved. The purpose of an area map is to orient the project geographically with other regional features. The area map should be large enough to identify river systems and tributaries.

Vicinity maps are needed to identify the detailed characteristics of the reach under consideration. There should be sufficient length of river reach included on the vicinity map to enable identification of stream type and to locate river meanders, sand bars and braided channels. The map should show coarse contours and relief. Intakes for municipal and industrial water, diversions for irrigation and power, and navigation channels should be clearly identified. Recreational areas such as camping and picnic grounds, bathing beaches, and recreatio boat docks should be identified. Cultivated areas and urban and industrial areas in the vicinity of towns and cities should be noted on the map. The direction of river flow should of course be clearly indicated.

It is highly desirable that aerial photographs of the study reach be obtained. Modern multi-image cameras use different ranges of the light spectrum to assist in identifying various features such as sewer outfalls, groundwater inflows, types of vegetation, sizes and heights of sandbars, river thalweg, river controls and geologic formations, existing bank protection works, old meander channels, and other features. Detailed contours can be developed from aerial photographs where such information is not readily available from topographic maps. Conditions of the river channel in the reach of concern are easy to record photographically, and such pictures can be very helpful in analysis of the river reach. Vegetation on floodplains, and seasonal variations of vegetation should be recorded photographically. Notable geologic formations should be photographed as well and supplemented with explanatory notes.

8.2.2 Geologic Data

The geophysical features of the study area are indicated by the geologic data. A geologic vicinity map, on which geophysical features are indicated is a basic need. The basic rock formations, outcroppings, and glacial and river deposits which form control points are valuable in an analysis of rivers. Soil type has important effects on sediment transport, infiltration rates, and groundwater flows. Channel geometry and roughness are also important factors in river mechanics and can be derived or inferred from geologic maps. Soil survey maps with engineering interpretations are available for a significant proportion of the United States. These may be helpful in determining sediment yields and sediment availability to the streams under consideration. Maps which trace the geologic history of a region are also useful in interpreting the evolution of channel patterns and river characteristics.

8.2.3 Hydrologic Data

Hydrologic data is required to determine the stream discharge, flood magnitudes, and duration and frequencies of floods preparatory to an analysis of river behavior and design of hydraulic structures. The basic data needed are stream discharge and stage data, historical floods, and highwater marks. Records of highwater period duration and estimates of flood discharge are valuable in flood-frequency analysis.

A flood-frequency curve can be prepared from recorded stream flow data and augmented by estimated discharges (using Manning's equation or its equivalent) from high water marks. Several methods ranging from sophisticated physical and stochastic analysis to simple approaches have been developed. The greatest difficulty in constructing a flood-frequen curve is lack of sufficient data. Approximate methods for extrapolatin the range of flood-frequency curves are available but are not discussed in detail here. (See Guidelines for Hydrology (Task Force on Hydrology and Hydraulics, 1973) for references).

A simple graphical method based on extreme value theory is reasonably satisfactory. The method consists of ordering the annual peak flood discharges of record from the largest to smallest, regardless of chronological order. The annual (flood) discharge is plotted against its recurrence interval on special probability (Gumbel, or others) pape The recurrence interval, RI is calculated from

$$RI = \frac{n+1}{m} \tag{8.1}$$

in which n is the number of years of record, and m is the order (largest flood is ranked 1) of the flood magnitude. Thus the highest flood discharge would have a recurrence interval of $n+1$ years and lowest would have a recurrence interval of $(1 + \frac{1}{n})$ years. The U.S. Wate

Resources Council (1967) has adopted the log-Pearson III distribution for use as a base method for determining flood flow frequencies. Details of the method and plotting paper may be obtained from the U. S. Geological Survey or the Federal Highway Administration (Washington or regional offices).

When adjusting discharge records from a gaging station to a specific reach, the flood peaks are prorated on the basis of drainage area ratios. Depending on drainage basin characteristics the exponent of the ratio varies from 0.5 to 0.8. Slope-area calculations for peak discharges are also used. In using this method, the conveyance of the channel is calculated using the Manning equation in which the roughness coefficient, n, needs to be estimated. An excellent reference relating n to channel conditions is presented pictorially in USGS Water Supply Paper 1849 (Barnes, 1967) which is published in book form. By referring to a catalog of (color) photographs, channel situations similar to a specific site can be identified and a relatively inexperienced individual can make a reliable estimate for n. The primary limitation of this catalog is that data are reported for only one discharge.

8.2.4 Hydraulic Data

Whenever possible, flow discharge, depth, velocity and sediment load data should be provided for river analysis. Direct measurement of flood flows should be made. To determine total flow rate, depth and velocity measurements need to be made at a sufficient number of subsections in a cross section. Discharge measurements made at various stages at a gaging station are essential for developing a stage-discharge rating curve.

Bed-material load, suspended load and wash load data may be obtained for some rivers in the water supply papers published by the U. S. Geological Survey, Corps of Engineers documents, state engineers' reports, or flood control and other water resources investigation reports. Information may also be obtained by direct sampling of the river.

Riverbed cross sections and profiles can be obtained from hydrographic surveys and are useful in sediment transport and backwater studies. In many instances, it is also helpful to know water temperatu

Observations of high-water marks along the river reach should be made. Each high-water mark and relevant profile should be established. These are helpful in calculating historical flood discharges. Also, stages achieved by ice jams at specific locations should be noted. Flood duration, debris in the river, distribution of flow, and magnitude of scour are useful information.

Channel changes which have occurred after floods are of particular interest in evaluating future effects on channel planform. Whenever possible, historic aerial photographs or equivalent maps which show river channels should be obtained.

8.2.5 Environmental Data

In making an environmental impact analysis of natural and developme activities on streams and rivers, it is necessary to obtain water quality and biological data for the streams. Municipal water and sewage treatment facilities and industrial plants utilizing river water should have recent records regarding river water quality which will be helpful in making an environmental analysis. Water quality data for certain rivers can be obtained from the U. S. Geological Survey. Information regarding turbidity, chemical quality, water temperature, wildlife, fish and fish habitat in rivers, and species of trees and other vegetation is essential to a comprehensive environmental analysis.

8.2.6 Climatologic Data

Stream gaging stations have been established on many streams throughout the United States. However there are some streams where, either a gaging station does not exist near the reach in question, or a gaging station does not exist at all. In such cases, it is necessary to estimate flood flows. These estimates may be based on regionalized estimating procedures or other prediction models using meteorological and watershed data inputs. These meteorological data are available from the National Weather Service (NWS) Data Center of the National Oceanic and Atmospheric Administration (NOAA), and estimates of average conditions can be made from rainfall data published by the NWS. Temperature records are helpful in making snowmelt estimates, and wind data are helpful in making wave height estimates on rivers, lakes and reservoirs as well as for coastal areas.

8.2.7 Checklist for Data Needs

The types of data needed for river analysis and the relative importance of each data type are listed in Table 8-1. Data with a degree of importance "Primary" are basic data required for any geomorphic, hydraulic, and environmental study of a river such as the Upper Mississippi. Whenever possible, these data should be directly collected from the field. Other data with a degree of importance "Secondary" are also very helpfu in an analysis of river but are considered a secondary requirement.

Limited data synthesis using records of surrounding stations and/or empirical, experimental and theoretical relations is possible. The quality of synthesis is also described as good, fair, or poor in Table 8-1. It is apparent that hydrologic and hydraulic data can be more readily synthesized than other types of data.

Data needs are, of course, a function of geologic region, river type, and the detail desired in the study. The data needs for general and detailed geomorphic and hydraulic studies of a river such as the Upper Mississippi are summarized below:

1. The topographic and hydrographic data for a general study should be taken along the river channel with spacing of 0.5 to 1.0 mile, except in subreaches having major channel changes such as upstream and downstream of locks and dams and of confluences. In these cases, the data at sections bracketing the location of major changes should be taken. Topographic and hydrographic surveys made before and after man-induced activities are very useful for assessing the river response to these activities. For a detailed study a 0.1 mile spacing of topographic and hydrographic data is necessary to provide more information regarding channel geometry.

2. The variations in stage and discharge in the Upper Mississippi River are relatively small. For both a general study and a detailed study daily discharge and stage are generally adequate to represent the flow characteristics in a river reach. The spacings between the discharge and the stage gaging stations are chosen to be 50 and 5 miles, respectively. These distances are generally adequate if the effects of infiltration, evaporation and local disturbance are small. However, stage and discharge data should be collected at river sections of special interest. For a detailed study, flow and channel characteristics related to regional or local disturbance should be recorded. Such records may include backwater curves, local scour, velocity and direction of flow, and other significant changes caused by bridge encroachment, dikes, bank revetment, dredging, locks and dams, and other structures.
3. Whenever possible, sediment load data should be provided for a general study of river response. There are a large number of theoretical, empirical and experimental relations formulated to determine the sediment load. However, no relation yields reliable results for general use unless verified using field data. Field data including size distributions of bed and bank material, bed load and

suspended load should be obtained at about 50 mile increments along the river to permit evaluation of the sediment transport function. For a detailed study the distance between the measuring stations should be small enough to reflect local changes of flow and local channel characteristics. Bed load, suspended load and wash load data on a weekly basis are desirable for both general and detailed studies.

4. The physical data and operational methods of regulating structures are very important for river analysis and should be provided for both general and detailed studies.
5. For both the general and detailed geomorphic study historic aerial photographs or equivalent maps which show river channels must be obtained to permit time lapsing of channel planform.
6. The environmental aspects of both the general and detailed study require at least seasonal data on forests, vegetation, and fish and wildlife habitat. Daily records of turbidity, water quality, and temperature are desirable.

Table 8-1 Checklist of Data Needs

Description of data or needed information	Degree of Data Importance	Quality of Synthesized Data
Maps and charts:		
Hydrographic	Primary	Poor
Topographic	Primary	Poor
Geologic	Secondary	Poor
Navigation charts	Secondary	-
Dredging surveys	Secondary	-
Potamology surveys	Secondary	-
County and city plats	Secondary	-
Aerial and other photos:		
Large scale photos of river and surrounding terrain	Primary	-
Small scale stereo pairs of river and surrounding terrain	Secondary	-
Color infrared photos for flow patterns, scour zones, and vegetation	Secondary	-
Ground photos	Secondary	-
Information on existing structures: locks, dams, dikes, diversions, or outfalls:		
Plans and details	Primary	-
Construction details	Secondary	-
Alterations and repairs	Secondary	-
Foundations	Secondary	-
Piers and abutments	Secondary	-
Scour	Primary	Fair
Aggradation	Primary	Fair
Field investigations	Primary	-
Hydraulics, Hydrology and Soils:		
Discharge records	Primary	Fair
Stage records	Primary	Fair
Flood frequency curves for stations near reach	Secondary	Fair
Flow duration curves (hydrographs)	Secondary	Fair
Newspaper, radio, television, accounts of large floods	Secondary	Fair

Table 8-1 Checklist of Data Needs (continued)

Description of data or needed information	Degree of Data Importance	Quality of Synthesized Data
Channel geometry		
Main channel	Primary	Fair
Side channel	Primary	Poor
Islands	Primary	Poor
Navigation channel	Secondary	Poor
Floodplain	Secondary	Poor
Slopes	Primary	Poor
Backwater calculation	Secondary	Poor
Bars	Secondary	Poor
Sinuosity	Secondary	Poor
Type (braided, meandering, straight)	Secondary	Poor
Controls (falls, rapids, restriction, rock outcropping dams, diversions)	Primary	Poor
Sediment discharge		
Size distribution	Primary	Fair
Bed and bank material sizes	Primary	Fair
Roughness coefficient n	Primary	Fair
Bed load	Primary	Fair
Suspended load	Primary	Fair
Wash load	Secondary	Fair
Ice:		
Recorded thickness	Secondary	Fair
Dates of freeze up and break up	Secondary	Good
Flow patterns and jams	Secondary	Good
Damage	Secondary	-
Regulating structures:		
Dams, diversions	Primary	-
Intake, outfalls	Secondary	-
Scour survey around hydraulic structures (piers, abutments, dikes)	Secondary	-
Planned and anticipated water resources projects	Primary	-
Lakes, tributaries, reservoirs or side channel impoundments	Primary	-
Field Surveys:		
Onsite inspections and photographs	Primary	-

Table 8-1 Checklist of Data Needs (continued)

Description of data or needed information	Degree of Data Importance	Quality of[1] Synthesized Data
Sample sediments	Secondary	-
Measure water and sediment discharge	Secondary	-
Observe channel changes or realignment since last maps or photos	Secondary	-
Identify high water lines or debris deposits due to recent floods	Secondary	-
Check magnitude of velocities and direction of flow	Secondary	-
Outcroppings	Secondary	-
Subsurface exploration	Secondary	-
Environmental data:		
Forests	Primary	Poor
Vegetation	Primary	Poor
Wildlife	Primary	Fair
Fish habitat	Primary	Fair
Turbidity	Primary	Fair
Chemical quality	Primary	Fair
Water temperature	Primary	Good
Climatologic data:		
National Weather Service records for precipitation	Secondary	Poor
Wind	Secondary	Poor
Temperatures	Secondary	Fair
Land use:		
Zoning maps	Secondary	-
Recent aerial photographs	Secondary	-
Planning committee records	Secondary	-
Urban areas	Secondary	-
Industrial areas	Secondary	-
Recreational areas	Secondary	-
Primitive areas	Secondary	-

8.3 Data Sources

The best data sources are national data centers where the principle function is to disseminate data. But it is usually necessary to collect data from a variety of other sources such as a field investigati interviews with local residents, and search through library materials of federal, state, and local agencies. The following list of sources is provided to serve as a guide to the data collection task:

Topographic Maps:

(1) Quadrangle maps -- U. S. Department of the Interior, Geological Survey, Topographic Division; and U. S. Department of the Army, Army Map Service.
(2) River plans and profiles -- U. S. Department of the Interior, Geological Survey, Conservation Division.
(3) National parks and monuments -- U. S. Department of the Interior, National Park Service.
(4) Federal reclamation project maps -- U. S. Department of the Interior, Bureau of Reclamation.
(5) Local areas -- commercial aerial mapping firms.
(6) American Society of Photogrammetry.

Planimetric Maps:

(1) Plats of public land surveys -- U. S. Department of the Interior, Bureau of Land Management.
(2) National forest maps -- U. S. Department of Agriculture, Forest Service.
(3) County maps -- State Highway Agency.
(4) City plats -- city or county recorder.
(5) Federal reclamation project maps -- U. S. Department of the Interior, Bureau of Reclamation.
(6) American Society of Photogrammetry.
(7) ASCE Journal -- Surveying and Mapping Division.

Aerial Photographs:

(1) The following agencies have aerial photographs of portions of the United States: U. S. Department of the Interior, Geological Survey, Topographic Division; U. S. Department of Agriculture, Commodity Stabilization Service, Soil Conservation Service and Forest Service; U. S. Air Force; various State agencies; commercial aerial survey; National Oceanic and Atmospheric Administration; and mapping firms.
(2) American Society of Photogrammetry.
(3) Photogrammetric Engineering
(4) Earth Resources Observation System (EROS)
Photographs from Gemini, Apollo, Earth Resources Technology Satellite (ERTS) and Skylab.

Transportation Maps:

(1) State Highway Agency.

Triangulation and Benchmarks:

(1) State Engineer.
(2) State Highway Agency.

Geologic Maps:

(1) U. S. Department of the Interior, Geologic Survey, Geologic Division; and State geological surveys or departments.
(Note -- some quadrangle maps show geologic data also).

Soils Data:

(1) County soil survey reports -- U. S. Department of Agriculture, Soil Conservation Service.
(2) Land use capability surveys -- U. S. Department of Agriculture, Soil Conservation Service.
(3) Land classification reports -- U. S. Department of the Interior, Bureau of Reclamation.
(4) Hydraulic laboratory reports -- U. S. Department of the Interior, Bureau of Reclamation.

Climatologic Data:

(1) National Weather Service Data Center.
(2) Hydrologic bulletin -- U. S. Department of Commerce, National Oceanic and Atmospheric Administration.
(3) Technical papers -- U. S. Department of Commerce, National Oceanic and Atmospheric Administration.
(4) Hydrometeorological reports -- U. S. Department of Commerce, National Oceanic and Atmospheric Administration, and U. S. Department of the Army, Corps of Engineers.
(5) Cooperative study reports -- U. S. Department of Commerce, National Oceanic and Atmospheric Administration and U. S. Department of the Interior, Bureau of Reclamation.

Stream Flow Data:

(1) Water supply papers -- U. S. Department of the Interior, Geological Survey, Water Resources Division.
(2) Reports of State engineers.
(3) Annual reports -- International Boundary and Water Commission, United States and Mexico.
(4) Annual reports -- various interstate compact commissions.
(5) Hydraulic laboratory reports -- U. S. Department of the Interior, Bureau of Reclamation.
(6) Corp of Engineers, U. S. Army, District offices.

Sedimentation Data:

(1) Water supply papers -- U. S. Department of the Interior, Geological Survey, Quality of Water Branch.
(2) Reports -- U. S. Department of the Interior, Bureau of Reclamation; and U. S. Department of Agriculture, Soil Conservation Service.
(3) Geological Survey Circulars -- U. S. Department of the Interior, Geological Survey.
(4) Corps of Engineers, U. S. Army, District offices, reservoir operation and dredging records.

Water Quality:

(1) Water supply papers -- U. S. Department of the Interior, Geological Survey, Quality of Water Branch.
(2) Reports -- U. S. Department of Health, Education, and Welfare, Public Health Service.
(3) Reports -- State public health departments.
(4) Water Resources Publications -- U. S. Department of the Interior, Bureau of Reclamation.
(5) Environmental Protection Agency, regional offices.
(6) State water quality agency.

Irrigation and Drainage Data:

(1) Agricultural census reports -- U. S. Department of Comme: Bureau of the Census.
(2) Agricultural statistics -- U. S. Department of Agricultu: Agricultural Marketing Service.
(3) Federal reclamation projects -- U. S. Department of the Interior, Bureau of Reclamation.
(4) Reports and Progress Reports -- U. S. Department of the Interior, Bureau of Reclamation.

Power Data:

(1) Directory of Electric Utilities -- McGraw Hill Publishin
(2) Directory of Electric and Gas Utilities in the United States -- Federal Power Commission.
(3) Reports -- various power companies, public utilities, State power commissions, etc.

Basin and Project Reports and Special Reports:

(1) U. S. Department of the Army, Corps of Engineers.
(2) U. S. Department of the Interior, Bureau of Land Managem Bureau of Mines, Bureau of Reclamation, Fish and Wildlif Service, and National Park Service.
(3) U. S. Department of Agriculture, Soil Conservation Servi
(4) U. S. Department of Health, Education, and Welfare, Publ: Health Service.
(5) State departments of water resources, departments of public works, power authorities and planning commissions
(6) Upper Mississippi River Basin Coordinating Committee.

Environmental Data:

(1) Sanitation and public health -- U. S. Department of Health, Education and Welfare, Public Health Service; State departments of public health.
(2) Fish and wildlife -- U. S. Department of the Interior, Fish and Wildlife Service; state game and fish departments.
(3) Municipal and industrial water supplies -- city water departments; State universities; Bureau of Business Research; State water conservation boards or State public works departments, State health agencies, Environmental Protection Agency, Public Health Service.
(4) Watershed management -- U. S. Department of Agriculture, Soil Conservation Service, Forest Service; U. S. Department of the Interior, Bureau of Land Management, Bureau of Indian Affairs.
(5) Upper Mississippi River Conservation Committee.

APPENDIX A

REFERENCES

Chapter 1

INTRODUCTION

Alexander, C. S. and Nunnally, N. R., 1972, "Channel Stability on the Lower Ohio River," Annals, Assoc. American Geog., Vol. 62, No. 3, September, pp. 411-617.

Coleman, J. M., 1969, "Brahmaputra River: Channel Processes and Sedimentation," Sedimentary Geology, Vol. 3, pp. 129-239.

Everitt, B. L., 1968, "Use of Cottonwood in the Investigation of the Recent History of a Floodplain," American Jour. Sci., Vol. 206, pp. 417-439.

Fisk, H. N., 1944, "Geological Investigation of the Alluvial Valley of the Lower Mississippi River," Mississippi River Commission, Vicksburg, Mississippi, 78 p.

Kolb, C. R., 1963, "Sediments Forming the Bed and Banks of the Lower Mississippi River and Their Effects on River Migration," Sedimentology, Vol. 2, pp. 227-234.

Lane, E. W., 1955, "The Importance of Fluvial Morphology in Hydraulic Engineering," ASCE Proc., Vol. 81, No. 745, 17 p.

Lathrop, D. W., 1968, "Aboriginal Occupation and Changes in River Channel on the Central Ucayali, Peru," American Antiquity, Vol. 33 pp. 62-79.

Leopold, L. B. and Maddock, T. Jr., 1953, "The Hydraulic Geometry of Stream Channels and Some Physiographic Implications," USGS Prof. Paper 252, 57 p.

Leopold, L. B. and Wolman, M. G., 1957, "River Channel Patterns: Braided, Meandering, and Straight," USGS Prof. Paper 282-B, 85 p.

Mackin, J. H., 1948, "Concept of the Graded River," Bulletin, Geologica Society of America, Vol. 59, May, pp. 463-512.

Ruppel, E., 1967, "Late Cenozoic Drainage Reversals, East and Central Idaho," Economic Geology, Vol. 62, pp. 648-663.

Santos-Cayudo, J. and Simons, D. B., 1972, "River Response," Environmen Impacts on Rivers, H. W. Shen, ed., Chap. 1, Water Resources Publ. Fort Collins, Colorado.

Schmudde, T. H., 1963, "Some Aspects of the Lower Missouri River Floodplain," Annals, Assoc. American Geog., Vol. 53, pp. 60-73.

Schumm, S. A., 1963, "The Disparity between Present Rates of Denudation and Orogeny," USGS Prof. Paper 454-H, 13 p.

Schumm, S. A., 1971, "Fluvial Geomorphology--The Historical Perspective," River Mechanics, H. W. Shen, ed., Vol. I, Chap. 4, Water Resources Publ., Fort Collins, Colorado.

Schumm, S. A. and Hadley, R. F., 1957, "Arroyos and the Semiarid Cycle of Erosion," American Jour. Sci., Vol. 255, pp. 161-174.

Schumm, S. A. and Lichty, R. W., 1963, "Channel Widening and Floodplain Construction along Cimarron River in Southwestern Kansas," USGS Prof. Paper 352-D, pp. 71-88.

Shull, C. A., 1922, "The Formation of a New Island in the Mississippi River," Ecology, Vol. 3, pp. 202-206.

Shull, C. A., 1944, "Observations of General Vegetational Changes on a River Island in the Mississippi River," American Midland Naturalist, Vol. 32, pp. 771-776.

Smith, H. T. U., 1940, "Notes on Historic Changes in Stream Courses of Western Kansas, with a Plea for Additional Data," Trans., Kansas Acad. Sci., Vol. 43, pp. 299-300.

Wallace, R., 1967, "Notes on Stream Channels Offset by the San Andreas Fault," Proc., Conference on Geologic Problems of the San Andreas Fault System, Stanford Univ. Pub. Geol. Sci.

Chapter 2

RIVER MECHANICS

Albertson, M. L., Barton, J. R., and Simons, D. B., 1960, "Fluid Mechanics for Engineers," Prentice-Hall Inc., New York.

Barnes, H. H., Jr., 1967, "Roughness Characteristics of Natural Channels," USGS Water-Supply Paper 1848, 213 p.

Beckman, E. W. and Furness, L. W., 1962, "Flow Characteristics of Elkhorn River near Waterloo, Nebraska," USGS Water Supply Paper, 1498-B, 34 p.

Brooks, N. M., 1958, "Mechanics of Streams with Movable Beds of Fine Sand," Trans., ASCE, Vol. 123, pp. 526-549.

Carnahan, B., Luther, H. A. and Wilkes, J. O., 1969, "Applied Numerical Methods," John Wiley and Sons, Inc., New York.

Carrey, W. C. and Keller, M. D., 1957, "Septematic Changes in the Beds of Alluvial Rivers," ASCE Hyd. Div. J., Vol. 83, No. HY4, 24 p.

Chen, Y. H., 1973, "Mathematical Modeling of Water and Sediment Routing in Natural Channels," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Chien, N., 1954, "Present Status of Research on Sediment Transport," Proc., ASCE, Vol. 80, No. 565, 33 p.

Chien, N., 1956, "Graphic Design of Alluvial Channels," Trans., ASCE, Vol. 121, pp. 1267-1280.

Chow, V. T., 1959, "Open Channel Hydraulics," John Wiley and Sons Inc., New York.

Colby, B. R., 1957, "Relationship of Unmeasured Sediment Discharge to Mean Velocity," Trans., AGU, Vol. 38, No. 5, October, pp. 708-719.

Colby, B. R., 1960, "Discontinuous Rating Curves for Pigeon Roost and Cuffaw Creeks in Northern Mississippi," USDA, Agr. Research Service Report, 41-46, 31 p.

Colby, B. R., 1961, "Effect of Depth of Flow on Discharge of Bed Material," USGS Water Supply Paper 1498-D, 12 p.

Colby, B. R., 1964, "Discharge of Sands and Mean-Velocity Relationships in Sand-Bed Streams," USGS Prof. Paper 462-A, 47 p.

Colby, B. R. and Hubbell, C. H., 1955, "Computation of Total Sediment Discharge, Niobrara River near Cody, Nebraska," USGS Water Supply Paper 1357, 187 p.

Corps of Engineers, 1968, "Missouri River Channel Regime Studies," MRD Sediment Series No. 13A, Omaha, Nebraska.

Culbertson, J. K. and Dawdy, D. R., 1964, "A Study of Fluvial Characteristics and Hydraulic Variables, Middle Rio Grande, New Mexico," USGS Water Supply Paper 1498-F, 74 p.

Dawdy, D. R., 1961, "Depth-Discharge Relations of Alluvial Streams--Discontinuous Rating Curves," USGS Water Supply Paper 1498-C, 16 p.

Einstein, H. A., 1950, "The Bed-Load Function for Sediment Transportatio in Open Channel Flows," USDA Technical Bulletin No. 1026.

Einstein, H. A., 1965, "Sedimentation--Part II: River Sedimentation," Handbook of Applied Hydrology, V. T. Chow, ed., Section 17, McGraw-Hill Book Co., New York.

Einstein, H. A. and Chien, N., 1956, "Similarity of Distorted River Models with Movable Beds," Trans., ASCE, Vol. 121, pp. 440-457.

Gessler, J., 1971, "Beginning and Ceasing of Sediment Motion," River Mechanics, H. W. Shen, ed., Vol. I, Chap. 7, Water Resources Publ., Fort Collins, Colorado.

Gessler, J., 1971, "Modeling of Fluvial Processes," River Mechanics, H. W. Shen, ed., Vol. II, Chap. 21, Water Resources Publ., Fort Collins, Colorado.

Gilbert, G. K., 1914, "The Transportation of Debris by Running Water," USGS Prof. Paper 86, 263 p.

Graf, W. H., 1971, "Hydraulics of Sediment Transport," McGraw-Hill Book Co., New York.

Guy, H. P., 1969, "Laboratory Theory and Methods for Sediment Analysis," USGS Techniques of Water Resources Investigation, Book 5, Chap. C1.

Guy, H. P., 1970, "Fluvial Sediment Concepts," USGS Techniques of Water Resources Investigations, Book 3, Chap. C1.

Harms, J. C. and Fahnestock, R. K., 1965, "Stratification, Bed Forms, and Flow Phenomena (with an example from the Rio Grande)," Primary Sedimentary Structures and Their Hydrodynamic Interpretation, Society of Economic Paleontologists and Mineralogists Special Publication No. 12.

Henderson, F. M., 1966, "Open Channel Flow," The McMillan Company, New York.

Hubbell, D. W. and Matekja, D. Q., 1959, "Investigations of Sediment Transportation, Middle Loup River at Dunning, Nebraska," USGS Water Supply Paper 1476, 123 p.

Inglish, C. C., 1949, "The Behavior and Control of Rivers and Canals," Res. Publ., Poona (india), No. 13, 2 Vols.

Ippen, A. T. and Dawson, J. H., 1951, "Design of Channel Contractions," Trans., ASCE, Vol. 116, pp. 326-346.

Ippen, A. T. and Drinker, P. A., 1962, "Boundary Shear Stresses in Curved Trapezoidal Channels," ASCE Hyd. Div. J., Vol. 88, No. HY5, September, pp. 143-179.

Kalinske, A. A., 1942, "Role of Turbulence in River Hydraulics," Hydraulic Conference, Iowa City, 2:226-79, Iowa University, Studies in Engineering, Bulletin 27.

Karaki, S., Mahmood, K., Richardson, E. V., Simons, D. B. and Stevens, M. A., 1974, "Highways in the River Environment, Hydraulic and Environmental Design Considerations," Report prepared for the Federal Highway Administration, Colorado State University, Fort Collins, Colorado.

Karman, T. von, 1930, "Mechanische Änlichkeit und Turbulenz," Nach. Ges. Wiss. Göttingen, Math. Phys. Kalsse, Vol. 58, and Proc. 3rd International Cong. Appl. Mech., Stockholm, Pt. I, Vol. 85.

Kennedy, J. F., 1963, "The Mechanics of Dunes and Antidunes in Erodible-Bed Channels," Fluid Mechanics Jour., Vol. 16, Pt. 4, pp. 521-544.

Knapp, R. T., 1951, "Design of Channel Curves for Supercritical Flow," Trans., ASCE, Vol. 116, pp. 296-325.

Lane, E. W. and Borland, W. M., 1951, "Estimating Bed Load," Trans., AGU, Vol. 32, No. 1, February, pp. 121-123.

Lane, E. W., 1957, "A Study of the Shape of Channels Formed by Natural Streams Flowing in Erodible Material," Missouri River Division Sediment Series No. 9, U.S. Army Engineer Division, Missouri River, Corps of Engineers, Omaha, Nebraska.

Lane, E. W. and Borland, W. M., 1954, "River-Bed Scour During Floods," Trans., ASCE, Vol. 119, pp. 1069-1079.

Laursen, E. M., 1958, "Sediment-Transport Mechanics in Stable-Channel Design," Trans., ASCE, Vol. 123, Paper 2918, pp. 195-203.

Laursen, E. M., 1963, "Analysis of Relief Bridge Scour," ASCE Hyd. Div. J., Vol. 89, No. HY3, May, pp. 93-118.

Leopold, L. B. and Wolman, M. G., 1960, "River Meanders," Bulletin, Geological Society of America, Vol. 71, June, pp. 769-794.

Leopold, L. B. and Langbein, W. B., 1962, "The Concept of Entropy in Landscape Evolution," USGS Prof. Paper 500A, 20 p.

Mahmood, K. and Ponce, V. M., 1975, "Computer Program for Sediment Transport, Documentation and Listing," Technical Report, Colorado State University, Fort Collins, Colorado.

Mahmood, K. and Shen, H. W., 1971, "Regime Concept of Sediment-Transporting Canals and Rivers," River Mechanics, H. W. Shen, ed., Vol. II, Chap. 30, Water Resources Publ., Fort Collins, Colorado.

Meyer-Peter, E. and Muller, R., 1948, "Formulas for Bed-Load Transport," Proc. of 2nd Congress, IAHR, Appendix 2, June 7-9, pp. 39-64.

Prandtl, L., 1925, "Über Die Ausgebildete Turbulenz," ZAMM 5, and Proc. 2nd Internatl. Cong. Appl. Mech., Zürich (1926).

Rana, S. A., Simons, D. B. and Mahmood, D., 1973, "Analysis of Sediment Sorting in Alluvial Channels," ASCE Hyd. Div. J., Vol. 99, No. HY11, November, pp. 1967-1980.

Raudkivi, A. J., 1967, "Loose Boundary Hydraulics," Pergamon Press, Oxford.

Ritter, J. R. and Helley, E. J., 1968, "An Optical Method for Determinin Particle Sizes of Coarse Sediment," USGS, Open-file report, 43 p.

Rouse, H., 1946, "Elementary Mechanics of Fluids," John Wiley and Sons, Inc., New York.

Rouse, H., Editor, 1950, "Engineering Hydraulics," John Wiley and Sons, Inc., New York.

Rouse, H., Bhoota, B. V. and Hsu, E. Y., 1951, "Design of Channel Expansions," Trans., ASCE, Vol. 116, pp. 347-363.

Rozovskii, I. L., 1957, "Flow of Water in Bends of Open Channels," Academy of Sciences, Ukrainian S.S.R., Kieve, USSR Translated from Russian. Published for NSF, Washington, D.C. and the Dept. of the Interior by the Israel Program for Scientific Translations.

Rzhanitsyn, N. A., 1960, "Morphological and Hydrological Regularities of the Structure of the River Net," Translated by D. B. Krimgold, Hydraulic Engineer, for Soil and Water Conservation Research Division, ARS, USDA and Water Resources Div., Geological Survey, U.S. Dept. of the Interior, original published Gidrometeoizdat Leningrad.

Schumm, S. A., 1960, "The Shape of Alluvial Channels in Relation to Sediment Type," USGS Prof. Paper 352-B, 30 p.

Schumm, S. A., 1963, "A Tentative Classification of Alluvial River Channels," USGS Circular 477, 10 p.

Schumm, S. A., 1971, "Fluvial Geomorphology--The Historical Perspective," River Mechanics, H. W. Shen, ed., Vol. I, Chap. 4, Water Resources Publ., Fort Collins, Colorado.

Shen, H. W., Editor, 1971, "River Mechanics," Vols. I and II, Water Resources Publ., Fort Collins, Colorado.

Shen, H. W., Editor, 1971, "Sedimentation," Water Resources Publ., Fort Collins, Colorado.

Sheppard, J. R., 1960, "Investigation of Meyer-Peter, Muller Bedload Formulas," U.S. Dept. of the Interior, Bureau of Reclamation, Sedimentation Section, Hydrology Branch, Division of Project Investigation, Denver, Colorado.

Shukry, A., 1950, "Flow Around Bends in Open Flume," Trans., ASCE, Vol. 115, pp. 751-779.

Simons, D. B. and Albertson, M. L., 1963, "Uniform Water Conveyence Channels in Alluvial Material," Trans., ASCE, Vol. 128, pp. 65-106.

Simons, D. B. and Richardson, E. V., 1963, "A Study of Variables Affecting Flow Characteristics and Sediment Transport in Alluvial Channels," Proc., Federal Inter-Agency Sedimentation Conference, Jackson, Mississippi, pp. 193-206.

Simons, D. B. and Richardson, E. V., 1966, "Resistance to Flow in Alluvial Channels," USGS Prof. Paper 422-J, 61 p.

Simons, D. B. and Richardson, E. V., 1971, "Flow in Alluvial Sand Channels," River Mechanics, H. W. Shen, ed., Vol. I, Chap. 9, Water Resources Publ., Fort Collins, Colorado.

Simons, D. B., Schumm, S. A., Stevens, M. A., Chen, Y. H. and Lagasse, P. F., 1975, "A Geomorphic Study of Pools 24, 25, and 26 in the Upper Mississippi and Lower Illinois Rivers," Colorado State University, Technical Report for the Waterways Experiment Station, Vicksburg, Mississippi.

Task Committee on Sediment Manual, 1971, "Discharge Formula," ASCE Hyd. Div. J., Vol. 97, No. HY4, April, pp. 523-567.

Tiffany, J. B., 1950, "Turbulence in the Mississippi River," Potamology Report No. 10-2, U.S. Army Waterways Experiment Station, C.E., Vicksburg, Mississippi.

U.S. Bureau of Reclamation, 1960, "Investigation of Meyer-Peter, Muller Bedload Formulas," Sedimentation Section, Hydrology Branch, Division of Project Investigations, Denver, Colorado.

Chapter 3

RIVER MORPHOLOGY

Anding, M. G., 1966, "Potamology Studies, Hydraulic Analysis of Mississippi River Channels," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 4, Chap. III.

Blench, T., 1969, "Mobile-Bed Fluviology," The University of Alberta Press, Edmonton, Alberta, Canada.

Chen, Y. H., 1973, "Mathematical Modeling of Water and Sediment Routing in Natural Channels," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Colby, B. R., 1964, "Discharge of Sands and Mean-Velocity Relationships in Sand-Bed Streams," USGS Prof. Paper 462A, 47 p.

Culbertson, D. M., Young, L. E. and Brice, J. C., 1967, "Scour and Fill in Alluvial Channels," USGS Open-file report, 58 p.

Edgar, D. E., 1973, "Geomorphic and Hydraulic Properties of Laboratory Rivers," Thesis submitted in partial fulfillment of the requirements for the degree of Master of Science, Colorado State Universit Fort Collins, Colorado.

Einstein, H. A., 1950, "The Bedload Function for Sediment Transportation in Open Channel Flows," USDA Technical Bulletin No. 1026.

Fairbridge, R. W., Editor, 1968, "Encyclopedia of Geomorphology," Reinhold Book Corp., New York.

Friedkin, J. F., 1945, "A Laboratory Study of the Meandering of Alluvial Rivers," Waterways Experiment Station, Vicksburg, Mississippi.

Gessler, J., 1971, "Modeling of Fluvial Processes," River Mechanics, H. W. Shen, ed., Vol. II, Chap. 21, Water Resources Publ., Fort Collins, Colorado.

Gilbert, G. K., 1914, "The Transportation of Debris by Running Water," USGS Prof. Paper 86, 263 p.

Haas, R. H., 1963, "Stabilization Problems, Lower Mississippi River," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 1, Chap. 1.

Henderson, F. M., 1961, "Stability of Alluvial Channels," ASCE Hyd. Div. J., Vol. 87, No. HY6, November, pp. 109-138.

Hooke, R. L., 1974, "Shear-Stress and Sediment Distribution in a Meander Bend," UNGI RAPPORT30, University of Uppsala, Dept. of Physical Geography, Uppsala, Sweden.

Khan, H. R., 1971, "Laboratory Study of Alluvial River Morphology," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Lagasse, P. F., 1975, "Interaction of River Hydraulics and Morphology with Riverine Dredging Operations," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Lane, E. W., 1955, "The Importance of Fluvial Morphology in Hydraulic Engineering," Proc., ASCE, Vol. 21, No. 745, 17 p.

Lane, E. W., 1957, "A Study of the Shape of Channels Formed by Natural Streams Flowing in Erodible Material," Missouri River Division Sediments Series No. 9, U.S. Army Engineer Division, Missouri River, Corps of Engineers, Omaha, Nebraska.

Lane, E. W. and Borland, W. M., 1954, "River-Bed Scour During Floods," Trans., ASCE, Vol. 119, pp. 1069-1079.

Leliavsky, S., 1966, "An Introduction to Fluvial Hydraulics," Dover Publications, Inc., New York.

Leopold, L. B. and Maddock, T., 1953, "The Hydraulic Geometry of Stream Channels and Some Physiographic Implications," USGS Prof. Paper 252, 57 p.

Leopold, L. B. and Wolman, M. G., 1957, "River Channel Patterns: Braided, Meandering, and Straight," USGS Prof. Paper 282-B, 85 p.

Leopold, L. B., Wolman, M. G. and Miller, J. P., 1964, "Fluvial Processes in Geomorphology," W. H. Freeman and Company, San Francisco.

Mackin, J. H., 1948, "Concept of the Graded River," Bulletin, Geologic Society of America, Vol. 59, May, pp. 463-512.

Maddock, T., 1969, "The Behavior of Straight Open Channels with Movable Beds," USGS Prof. Paper 622-A, 70 p.

Mathes, G. H., 1941, "Basic Aspects of Stream-Meanders," Trans., AGU, pp. 632-636.

Morisawa, M., 1968, "Streams, Their Dynamics and Morphology," McGraw-Hill Book Company, New York.

Netherlands Engineering Consultants (NEDECO), 1959, "River Studies and Recommendations for Improvement of Niger and Benue," North Holland Publishing Company, The Hague, Amsterdam.

Nordin, C. F., 1964, "Aspects of Flow Resistance and Sediment Transport, Rio Grande near Bernalillo, New Mexico," USGS Water Supply Paper 1498-H, 41 p.

Ockerson, J. A., 1898, "Dredges and Dredging on the Mississippi River," Trans., ASCE, Vol. XL, pp. 215-310.

Oglesby, R. J., Carlson, C. A. and McCann, J. A., 1971, "River Ecology and Man," Academic Press, New York.

Rubey, W. W., 1952, "Geology and Mineral Resources of the Hardin and Brussels Quadrangles (In Illinois)," USGS Prof. Paper 218, 179 p.

Santos-Cayudo, J., 1972, "Stage Determination for High Discharges," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Dept. of Civil Engineering, Colorado State University, Fort Collins, Colorado.

Santos-Cayudo, J. and Simons, D. B., 1972, "River Response," Environment Impacts on Rivers, H. W. Shen, ed., Chap. 1, Water Resources Publ., Fort Collins, Colorado.

Schumm, S. A., 1969, "River Metamorphosis," ASCE Hyd. Div. J., Vol. 95, No. HY1, January, pp. 255-273.

Schumm, S. A., 1971, "Fluvial Geomorphology--The Historical Perspective," River Mechanics, H. W. Shen, ed., Vol. I, Chap. 4, Water Resources Publ., Fort Collins, Colorado.

Schumm, S. A., 1972, "River Morphology," Dowden, Hutchison, and Ross, Inc., Strousburg, Pennsylvania.

Schumm, S. A. and Khan, H. R., 1972, "Experimental Study of Channel Patterns," Bulletin, Geologic Society of America, Vol. 83, June, pp. 1755-1770.

Shen, H. W., Editor, 1971, "Environmental Impact on Rivers," Water Resources Publ., Fort Collins, Colorado.

Shulits, S., 1941, "Rational Equation of River-Bed Profile," Trans., AGU, Vol. 22, pp. 622-630.

Shull, C. A., 1922, "The Formation of a New Island in the Mississippi River," Ecology, Vol. 3, pp. 202-206.

Simons, D. B., Schumm, S. A. and Stevens, M. A., 1974, "Geomorphology of the Middle Mississippi," Colorado State University, Technical Report for the Waterways Experiment Station, Vicksburg, Mississippi.

Winkley, B. R., Simons, D. B. and Schumm, S. A., 1972, "Rivers as Dynamic Systems," ASCE Task Committee Paper.

Chapter 4

THE GEOMORPHIC PERSPECTIVE

Corps of Engineers, Rock Island District, 1974, "Environmental Impact Statement (Draft)--Upper Mississippi River 9-Foot Channel Operations and Maintenance," 12 Vols.

Corps of Engineers, St. Paul District, 1974, "Final Environmental Impact Statement--Operation and Maintenance, 9-Foot Navigation Channel, Upper Mississippi River, Head of Navigation to Gutenburg, Iowa," 2 Vols.

Dorsey, F. L., 1941, "Master of the Mississippi," Riverside Press, Cambridge, Massachusetts, Houghton Mifflin, Co., Boston.

Fisk, H. N., 1947, "Fine-Grained Alluvial Deposits and Their Effects on Mississippi River Activity," 2 Vols., Waterways Experiment Station, Vicksburg, Mississippi.

Frye, J. C., Willman, H. B. and Black, R. F., 1965, "Outline of Glacial Geology of Illinois and Wisconsin," Wright, H. E., Jr., and Frye, D. G., eds., The Quarternary of the United States, Princeton University Press, Princeton, New Jersey, pp. 43-61.

Fuller, M. L., 1912, "New Madrid Earthquake," USGS Bulletin 494, 119 p.

Holly, F. M., Simons, D. B. and Chen, Y. H., 1974, "Hydrologic Study--Saint Maurice Towhead Coal Fired Station--Gulf States Utilities Company," Technical Report for Stone and Webster Engineering Company, Colorado State University, Fort Collins, Colorado.

Humphreys, A. A. and Abbot, H. L., 1876, "Report upon the Physics and Hydraulics of the Mississippi River," Prof. Paper of the Corps of Engineers, No. 13, 1861, Reprint 1876.

Lagasse, P. F., 1975, "Interaction of River Hydraulics and Morphology with Riverine Dredging Operations," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Lane, E. W., 1957, "A Study of the Shape of Channels Formed by Natural Streams Flowing in Erodible Material," Missouri River Division Sediment Series No. 9, U.S. Army Engr. Div., Missouri River, Corps of Engineers, Omaha, Nebraska.

Mack, F. J., 1970, "Sediment Yields in the Upper Mississippi River Basin," Proc. of a Seminar on Sediment Transport in Rivers and Reservoirs, Paper No. 4, Corps of Engineers, Hydrologic Engineering Center, Davis, California.

Ockerson, J. A., 1898, "Dredges and Dredging on the Mississippi River," Trans., ASCE, Vol. XL, pp. 215-310.

Rubey, W. W., 1952, "Geology and Mineral Resources of the Hardin and Brussels Quadrangles (In Illinois)," USGS Prof. Paper 218, 179 p.

Simons, D. B., Schumm, S. A. and Stevens, M. A., 1974, "Geomorphology of the Middle Mississippi," Colorado State University, Technical Report for the Waterways Experiment Station, Vicksburg, Mississippi.

Simons, D. B., Schumm, S. A., Stevens, M. A., Chen, Y. H. and Lagasse, P. F., 1975, "A Geomorphic Study of Pools 24, 25, and 26 in the Upper Mississippi and Lower Illinois Rivers," Colorado State University, Technical Report for the Waterways Experiment Station, Vicksburg, Mississippi.

Thornbury, W. D., 1965, "Regional Geomorphology of the United States," John Wiley and Sons, New York.

Twain, M., 1968, "Life on the Mississippi," Washington Square Press, Simon and Schuster, New York.

Upper Mississippi River Basin Coordinating Committee, 1972, "Upper Mississippi River Comprehensive Basin Study (UMRCBS)," Vols. I, II, III, V, and VI.

Zumberge, J. H., 1952, "The Lakes of Minnesota--Their Origin and Classification," University of Minnesota, Minnesota Geological Survey Bulletin 35, Minneapolis, University of Minnesota Press.

Chapter 5

RIVER RESPONSE

Armstrong, R. C., 1970, "Replacement--Lock and Dam No. 26: Plans Considered," Jour. Waterways and Harbors Division, ASCE, Vol. 96, No. WW1, February, pp. 49-64.

Chen, Y. H., 1973, "Mathematical Modeling of Water and Sediment Routing in Natural Channels," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Corps of Engineers, St. Paul District, 1974, "Final Environmental Impact Statement--Operation and Maintenance, 9-Foot Navigation Channel, Upper Mississippi River, Head of Navigation to Gutenburg, Iowa," 2 Vols.

Dass, P., 1975, "One-Dimensional Multi-Stream Flow Approach for Water and Sediment Routing in Natural Channels," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State Universtiy, Fort Collins, Colorado.

Decker, E. R., 1970, "Replacement--Lock and Dam 26: History, Objectives and Scope," Jour. Waterways, Harbors and Coastal Engineering Division, ASCE, Vol. 96, No. WW1, February, pp. 1-8.

Degenhardt, E. A., 1973, "Channel Stabilization of the Middle Mississippi River," Thesis submitted in partial fulfillment of the requirements for the degree of Master of Science, Colorado State University, Fort Collins, Colorado.

Jordan, P. R., 1968, "Summary and Analysis of Sediment Records in Relation to St. Louis Harbor Sedimentation Problems," USGS Open-file Report.

Lagasse, P. F., 1975, "Interaction of River Hydraulics and Morphology with Riverine Dredging Operations," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Lane, E. W., 1955, "The Importance of Fluvial Geomorphology in Hydraulic Engineering," Proc., ASCE, Vol. 81, Paper 795, pp. 1-17.

Lugn, A. L., 1927, "Sedimentation in the Mississippi River between Davenport, Iowa and Cairo, Illinois," Augstana Library Publications, No. 11.

Mack, F. J., 1970, "Sediment Yields in the Upper Mississippi River Basin," Proc. of a Seminar on Sediment Transport in Rivers and Reservoirs, Paper No. 4, Corps of Engineers, Hydrologic Engineering Center, Davis, California.

Rhodes, R., 1972, "The Mississippi," Playboy, Vol. 19, No. 12, December.

Rubey, W. W., 1952, "Geology and Mineral Resources of the Hardin and Brussels Quadrangles (In Illinois)," USGS Prof. Paper 218, 179 p.

Santos-Cayudo, J. and Simons, D. B., 1972, "River Response," Environmental Impacts on Rivers, H. W. Shen, ed., Chap. 1, Water Resources Publ., Fort Collins, Colorado.

Simons, D. B., 1975, "The Geomorphic and Hydraulic Response of Rivers," Presented to: North American Wildlife and Natural Resources Conference, March 16-19, 1975, Pittsburg, Pennsylvania.

Simons, D. B., Schumm, S. A. and Stevens, M. A., 1974, "Geomorphology of the Middle Mississippi," Colorado State University, Technical Report for the Waterways Experiment Station, Vicksburg, Mississippi

Simons, D. B., Schumm, S. A., Stevens, M. A., Chen, Y. H. and Lagasse, P. F., 1975, "A Geomorphic Study of Pools 24, 25, and 26 in the Upper Mississippi and Lower Illinois Rivers," Colorado State University, Technical Report for the Waterways Experiment Station, Vicksburg, Mississippi.

Toffaleti, F. B., 1970, "Mississippi River Dam No. 26 Sedimentation Study," Proc. of a Seminar on Sediment Transport in Rivers and Reservoirs, Paper No. 6, Corps of Engineers, Hydrologic Engineering Center, Davis, California.

Tuttle, J. R., 1970, "Sediment Problems in St. Louis Harbor," Proc. of Seminar on Sediment Transport in Rivers and Reservoirs, Paper No. 7, Corps of Engineers, Hydrologic Engineering Center, Davis, California.

Upper Mississippi River Basin Coordinating Committee, 1972, "Upper Mississippi River Comprehensive Basin Study (UMRCBS)," Vols. I, II, III, V, and VI.

Chapter 6

RIVER RESPONSE TO CHANNEL STABILIZATION

Ahmad, M., 1953, "Experiments on Design and Behavior of Spur Dikes," Proc., IAHR, ASCE Joint Meeting, University of Minnesota, August.

Bondurant, D. C., 1963, "Channel Stabilization on the Missouri River," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 1, Chap. IV.

Bondurant, D. C., 1970, "Basic Sedimentation Problems," Proc. of a Seminar on Sediment Transport in Rivers and Reservoirs, Paper No. 1, Corps of Engineers, Hydrologic Engineering Center, Davis, California.

Bureau of Reclamation, 1960, "Design of Small Dams," U.S. Govt. Printing Office, Washington, D.C.

California Division of Highways, 1970, "Bank and Shore Protection in California Highway Practice," Dept. of Public Works, November.

Carlson, E. J. and Dodge, E. A., 1962, "Control of Alluvial Rivers by Steel Jetties," Jour. Waterways and Harbors Division, ASCE, WW4, November, pp. 53-81.

Corps of Engineers, U.S. Army, 1959, "Earth Embankments," Manual EM 1110-2-2300, April, 36 p.

Corps of Engineers, U.S. Army, 1962, "Stability of Earth and Rockfill Dams," Manual EM 1110-2-1902, February, 67 p.

Corps of Engineers, U.S. Army, 1965, "Retaining Walls," Manual EM 1110-2-2502, January, 13 p.

Cress, G. D., 1973, "Stabilization of Alluvial Rivers," Thesis submitted in partial fulfillment of the requirements for the degree of Master of Science, Colorado State University, Fort Collins, Colorado.

Dass, P., 1975, "One-Dimensional Multi-Stream Flow Approach for Water and Sediment Routing in Natural Channels," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Degenhardt, E. A., 1973, "Channel Stabilization of the Middle Mississippi River," Thesis submitted in partial fulfillment of the requirements for the degree of Master of Science, Colorado State University, Fort Collins, Colorado.

Feagin, L. B., 1966, "Program of the Mississippi River Commission," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 4, Chap. V.

Fenwick, G. B., Editor, 1969, "State of Knowledge of Channel Stabilization in Major Alluvial Rivers," Technical Report No. 7, Corps of Engineers, Committee on Channel Stabilization.

Franco, J. J., 1970, "Insights Gained from River Sedimentation Models," Proc. of Seminar on Sediment Transport in Rivers and Reservoirs, Paper No. 11, Corps of Engineers, Hydrologic Engineering Center, Davis, California.

Friedkin, J. F., 1945, "A Laboratory Study of the Meandering of Alluvial Rivers," Waterways Experiment Station, Vicksburg, Mississippi.

Gill, M. A., 1972, "Erosion of Sand Beds around Spur Dikes," ASCE Hyd. Div. J., Vol. 98, No. HY9, September, pp. 1587-1602.

Haas, R. H., 1963, "Stabilization Problems, Lower Mississippi River," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 1, Chap. I.

Haas, R. H., 1964, "Regulation and Contraction Works, Alluvial Rivers," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 2, Chap. I.

Hartke, W. O., 1966, "The Stone-Fill Dike in the Middle Mississippi," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 4, Chap. II.

Henley, J. E., 1966, "Channel Stabilization Work in the Vicksburg District," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 4, Chap. I.

Hudson, R. Y., 1959, "Laboratory Investigations of Rubble-Mound Break-Waters," Jour. Waterways and Harbors Division, ASCE, Vol. 85, No. WW3, September, pp. 93-121.

Karaki, S., 1959, "Hydraulic Model Studies of Spur Dikes for Highway Bridge Openings," Report No. CER59-SSK36, Colorado State University (also Bull. 286, Highway Research Board, Washington).

Karaki, S., Mahmood, K., Richardson, E. V., Simons, D. B. and Stevens, M. A., "Highways in the River Environment, Hydraulic and Environmental Design Considerations," Report prepared for the Federal Highway Administration, Colorado State University, Fort Collins, Colorado.

Lagasse, P. F., 1975, "Interaction of River Hydraulics and Morphology with Riverine Dredging Operations," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Laursen, E. M., 1960, "Scour at Bridge Crossings," ASCE Hyd. Div. J., Vol. 86, No. HY2, February, pp. 39-54.

Laursen, E. M., 1963, "An Analysis of Relief Bridge Scour," ASCE Hyd. Div. J., Vol. 89, No. HY3, May, pp. 93-118.

Lindner, C. P., 1969, "Channel Improvement and Stabilization Measures," Technical Report No. 7, Corps of Engineers, Committee on Channel Stabilization.

Maher, T. F., 1963, "Degradation Study of the Middle Mississippi River, Vicinity of St. Louis, Missouri," Proc. of the Federal Inter-Agency Sedimentation Conference, Jackson, Mississippi, pp. 424-430.

Petersen, M. S., 1966, "Laboratory Contributions to Channel Stabilization," Jour. Waterways and Harbors Division, ASCE, Vol. 92, No. WW1, February, pp. 87-108.

Simons, D. B., Schumm, S. A. and Stevens, M. A., 1974, "Geomorphology of the Middle Mississippi," Colorado State University, Technical Report for the Waterways Experiment Station, Vicksburg, Mississippi

Stevens, M. A., 1969, "Scour in Riprap at Culvert Outlets," Dissertation submitted in partial fulfillment for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Stevens, M. A. and Simons, D. B., 1971, "Stability Analysis for Coarse Granular Material on Slopes," River Mechanics, H. W. Shen, ed., Vol. I, Chap. 17, Water Resources Publ., Fort Collins, Colorado.

Task Committee Progress Report, 1965, "Channel Stabilization of Alluvial Rivers," Jour. Waterways and Harbors Division, ASCE, Vol. 91, No. WW1, February, pp. 7-37.

Tiefenbrun, A. J., 1963, "Bank Stabilization of Mississippi River between the Ohio and Missouri Rivers," Proc. of the Federal Inter-Agency Sedimentation Conference, Jackson, Mississippi, pp. 387-398.

Tiffany, J. B., 1963, "Review of Research on Channel Stabilization of the Mississippi River 1931-1962," Technical Report No. 2, Corps of Engineers, Committee on Channel Stabilization.

Tiffany, J. B., 1963, "Recommended Research on Channel Stabilization," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 1, Chap. VII.

Winkley, B. R., 1971, "Practical Aspects of River Regulation and Control," River Mechanics, H. W. Shen, ed., Vol. I, Chap. 19, Water Resources Publ., Fort Collins, Colorado.

Woodson, R. C., 1961, "Stabilization of the Middle Rio Grande in New Mexico," Jour. Waterways and Harbors Division, ASCE, Vol. 87, No. WW4, November, pp. 1-15.

Chapter 7

RIVER RESPONSE TO DREDGING

Anderson, A. G., 1975, "A Proposal to Investigate the Feasibility of Riverine Disposal of Dredged Material Derived from the Navigation Channels in Meandering Rivers," University of Minnesota, St. Anthony Falls Hydraulic Laboratory.

Basco, D. R., 1973, "Systems Engineering and Dredging--The Feedback Problem," Sea Grant Publ. No. TAMU-SG-74-205, Dept. of Civil Engineering, Texas A & M University.

Basco, D. R., 1975, "Analytical Model of Hydraulic Pipeline Dredge," Jour. Waterways, Harbors and Coastal Engineering Division, ASCE, Vol. 101, No. WW1, February, pp. 33-48.

Biggs, R. B., 1968, "Environmental Effects of Overboard Spoil Disposal," Jour. Sanitary Engineering Division, ASCE, Vol. 94, No. SA3, June, pp. 477-487.

Black, W. L., 1973, "Introduction to Dredging," Estuarine Processes Seminar, Waterways Experiment Station, 7-11 May 1973.

Boyd, M. B., Saucier, R. T., Keeley, J. W., Montgomery, R. L., Brown, R. D., Mathis, D. B. and Guice, C. J., 1972, "Disposal of Dredge Spoil--Problem Identification and Assessment and Research Program Development," Technical Report No. H-72-8, Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi.

Breusers, H. N. C., Allersma, E. and van der Weide, J., 1968, "Hydraulic Model Investigations in Dredging Practice," Proc., World Dredging Conference 1968, (WODCON), pp. 92-116.

Bubenik, M. C., 1963, "Channel Stabilization, Columbia River," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 1, Chap. VI.

Cable, C. C., 1969, "Optimum Dredging and Disposal Practices in Estuaries," ASCE Hyd. Div. J., Vol. 95, No. HY1, January, pp. 103-114.

Carey, W. C., 1966, "Comprehensive River Stabilizaiton," Jour. Waterways and Harbors Division, ASCE, Vol. 92, No. WW1, February, pp. 59-86.

Chatley, H., 1927, "The Stability of Dredged Cuts in Alluvium," Jour. of Junior Institution of Engineers, London, Vol. 17, July, pp. 525-531.

Chatley, H., 1929, "Problems in the Theory of River Engineering," Selected Engineering Papers No. 71, The Institution of Civil Engineers, London.

Chatley, H., 1938, "River Flow Problems," Engineering--An Illustrated Weekly Journal, London, Vol. 146.

Corps of Engineers, Mobile District, 1970, "Environmental Investigations of Dredging Activities in Mobile Bay, Alabama."

Corps of Engineers, North Central Division, 1972, "Upper Mississippi River Navigation Charts."

Corps of Engineers, Portland District, 1973, "Study of Hopper Dredging Coastal Harbor Entrances and Columbia River Estuary Bars, States of Oregon and Washington."

Corps of Engineers, Rock Island District, 1974, "Environmental Impact Statement (Draft)--Upper Mississippi River 9-Foot Channel Operations and Maintenance," 12 Vols.

Corps of Engineers, St. Louis District, 1974, "Environmental Study--Locks and Dam No. 26 (Replacement), Upper Mississippi River Basin--Alton, Illinois," Harland Batholomew and Associates, St. Louis, Missouri.

Corps of Engineers, St. Paul District, 1935, "Report on Sedimentary Characteristics of the Upper Mississippi River."

Corps of Engineers, St. Paul District, 1974, "Final Environmental Impact Statement--Operation and Maintenance, 9-Foot Navigation Channel, Upper Mississippi River, Head of Navigation to Gutenburg, Iowa," 2 Vols.

Corps of Engineers, Waterways Experiment Station, 1941, "Model Study of Plans for Elimination of Shoaling in the Vicinity of the Manchester Islands, Ohio River," Technical Memo No. 181-1, Vicksburg, Mississippi.

Corps of Engineers, Waterways Experiment Station, 1962, "Development and Maintenance of Navigation Channel, Arkansas River, Arkansas and Oklahoma--Hydraulic Model Investigations," Technical Report No. 2-608, Vicksburg, Mississippi.

Corps of Engineers, Waterways Experiment Station, 1974, "First Annual Report-Dredged Material Research Program," Office of Dredged Material Research, Vicksburg, Mississippi.

Degenhardt, E. A., 1973, "Channel Stabilization of the Middle Mississippi River," Thesis submitted in partial fulfillment of the requirements for the degree of Master of Science, Colorado State University, Fort Collins, Colorado.

Edge, B. L., 1974, "Environmental Compatible Techniques for Dumping Dredge Spoil at Sea," Proc., World Dredging Conference V, (WODCON), pp. 721-760.

Eisenberg, W. S., 1973, "Environmental Aspects of Jacksonville Harbor Project," Jour. of Waterways, Harbors and Coastal Engineering Division, ASCE, Vol. 99, No. WW2, May, pp. 265-280.

Emerson, J. W., 1971, "Channelization: A Case Study," Science, Vol. 173, July 23, 1971, pp. 325-326.

Feagin, L. B., 1947, "Dredging," Draft prepared for the Committee on Regulation and Stabilization of Rivers by Open Channel Work, Chap. VI.

Franco, J. J. and McKellar, C. D., Jr., 1973, "Lock and Dam No. 8 Arkansas River Navigation Project--Hydraulic Model Investigation," Technical Report No. H-73-10, Waterways Experiment Station, Vicksburg, Mississippi.

Frijlink, H. C., 1972, "Dredging Projects in Developing Countries," Terre et Aqua, Internatl. Jour. on Public Works, Ports and Waterways Development, Internatl. Assoc. of Dredging Companies.

Gessler, J., 1970, "Self Stabilizing Tendencies of Alluvial Channels," Jour. Waterways and Harbors Division, ASCE, Vol. 96, No. WW2, May, pp. 235-250.

Gilbert, G., 1914, "The Transportation of Debris by Running Water," USGS Prof. Paper 86, 263 p.

Gower, G. L., 1967, "A History of Dredging," Proc. of the Institution of Civil Engineers Symposium, 18 October 1967, London.

Hakenjos, C. B., 1974, "The 1973 Mississippi River Flood and the Role of Hydraulic Dredging," Proc. of the Sixth Dredging Seminar, Texas A & M University, Center for Dredging Studies, Report No. CDS-176.

Hallmark, D. E. and Smith, G. L., 1965, "Stability of Channels by Armor Plating," Jour. Waterways and Harbors Division, ASCE, Vol. 91, No. WW3, Part 1, August, pp. 117-138.

Hammond, R., 1969, "Modern Dredging Practice," Frederick Muller Ltd., London.

Hann, R. W. and Slowey, J. F., 1972, "Texas Gulf Coast Sediment and Water Quality Management," Proc., World Dredging Conference 1972, (WODCON), pp. 577-598.

Harrison, J. E. and Chisholm, L. C., 1974, "Identification of Objectionable Environmental Conditions and Issues Associated with Confined Disposal Areas," Contract Report No. D-74-1, A. D. Little, Inc.

Harrison, W., 1967, "Environmental Effects of Dredging and Spoil Deposition," Proc., World Dredging Conference 1967, (WODCON), pp. 535-560.

Helm, D. R. and Boland, T. L., 1972, "Upper Mississippi River Natural Resources Bibliography," Upper Mississippi River Conservation Committee, Fish Technical Section.

Herbich, J. B., 1968, "Research Needs of Dredging Industry," Proc., World Dredging Conference 1968, (WODCON), pp. 698-710.

Herbich, J. B. and Schiller, R. E., 1974, "Environmental Effects of Dredging," Proc., World Dredging Conference V, (WODCON), pp. 699-719.

Herbich, J. B. and Snider, R. H., 1969, "Bibliography on Dredging," Sea Grant Publ. No. 203, Texas A & M University.

Hickson, R. E., 1965, "Columbia River Stabilization and Improvement for Navigation," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 3, Chap. IV.

Hopman, R. J., 1972, "Operational Planning and Its Associated Problems for Dredging in the Pacific Northwest," Proc., World Dredging Conference 1972, (WODCON), pp. 29-48.

Huston, J., 1970, "Hydraulic Dredging--Theoretical and Applied," Cornell Maritime Press, Cambridge.

Hyde, G. E. and Beeman, O., 1963, "Improvement of the Navigability of the Columbia River by Dredging and Constriction Works," Proc. of the Federal Inter-Agency Sedimentation Conference, Jackson, Mississippi, pp. 454-460.

Johnson, B. H., 1974, "Investigation of Mathematical Models for the Physical Fate Prediction of Dredged Material," Technical Report No. D-74-1, Office of Dredged Material Research, Waterways Experiment Station, Vicksburg, Mississippi.

Johnson, H. D., 1970, "Current Dutch Practice for Evaluating River Sediment Transport Processes," Proc. of a Seminar on Sediment Transport in Rivers and Reservoirs, Paper No. 15, Corps of Engineers, Hydrologic Engineering Center, Davis, California.

Jordan, P. R., 1968, "Summary and Analysis of Sediment Records in Relation to St. Louis Harbor Sedimentation Problems," USGS Open-file Report.

Kidby, H. A., 1966, "Columbia and Lower Willamette Rivers, 40-by 600-Foot Project," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 3, Chap. III.

Komura, S. and Simons, D. B., 1967, "River-Bed Degradation below Dams," ASCE Hyd. Div. J., Vol. 93, No. HY4, July, pp. 1-14.

Kondrat'ev, N. E. Lyapin, A. N., Popov, I. V., Pin'kovskii, S. I., Fedorov, N. N. and Yakunin, I. I., 1962, "River Flow and Channel Formation," The NSF and the Dept. of the Interior, Israel Program for Scientific Translations.

Lagasse, P. F., 1975, "Interaction of River Hydraulics and Morphology with Riverine Dredging Operations," Dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado.

Lane, E. W., 1937, "Stable Channels in Erodible Material," Trans., ASCE, Vol. 102, Paper 1957, pp. 123-132.

Lane, E. W., 1955, "Design of Stable Channels," Trans., ASCE, Vol. 120, Paper 2776, pp. 1234-1261.

Lee, G. F. and Plumb, R. H., 1974, "Literature Review on Research Study for the Development of Dredged Material Disposal Criteria," Contract Report No. D-74-1, Institute for Environmental Studies, University of Texas, Dallas.

Livesey, R. H., 1963, "Channel Armoring below Fort Randall Dam," Proc. of the Federal Inter-Agency Sedimentation Conference, Jackson, Mississippi, pp. 461-469.

Madden, E. B., 1964, "Channel Design for Modified Sediment Regime Conditions on the Arkansas River," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 2, Chap. III.

Maltby, F. B., 1905, "Hydraulic Dredging on the Mississippi River," Trans., ASCE, Vol. LIV, Part C, pp. 391-478.

Maximoff, S. P., 1904, "A Desirable Method of Dredging Channels Through River Bars," Trans., ASCE, Vol. LII, pp. 215-237.

Meccia, R. M., Allanoch, W. C., Jr. and Griffis, F. H., 1975, "Engineering Challenges of Dredged Material Disposal," Jour. of Waterways, Harbors and Coastal Engineering Division, ASCE, Vol. 101, No. WW1, February, pp. 1-14.

Melone, A. M., 1974, "Exclusion and Ejection of Sediment from Canals," Thesis submitted in partial fulfillment of the requirements for the degree of Master of Science, Colorado State University, Fort Collins, Colorado.

Mohr, A. W., 1974, "Development and Future of Dredging," Jour. of Waterways, Harbors and Coastal Engineering Division, ASCE, Vol. 100, No. WW2, May, pp. 69-85.

Morton, J. W., 1973, "A Selected Bibliography on Dredging and the Disposal of Dredge Spoils," Cornell University.

Murphy, W. L. and Zeigler, T. W., 1974, "Practices and Problems of the Confinement of Dredged Material in Corps of Engineers Projects," Technical Report D-74-2, Office of Dredged Material Research, Waterways Experiment Station, Vicksburg, Mississippi.

Netherlands Engineering Consultants (NEDECO), 1959, "River Studies and Recommendations for Improvement of Niger and Benue," North Holland Publishing Company, The Hague, Amsterdam.

Netherlands Engineering Consultants (NEDECO), 1965, "Development and Maintenance Dredging on Rivers--A Study on Hydraulic Engineering Considerations," The Hague, Holland.

Netherlands Engineering Consultants (NEDECO), 1973, "Rio Magdalena and Canal del Dique Survey Project," The Hague, Holland.

Nordin, C. F., 1971, "Graphical Aids for Determining Scour Depth in Long Contractions," Unpublished Note, Colorado State University, Fort Collins, Colorado.

Norgaard, R. B., 1971, "Streamflow and Sediment Deposition in the Lower Columbia," Water Resources Research, Vol. 7, No. 5, October, pp. 1199-1207.

Ockerson, J. A., 1898, "Dredges and Dredging on the Mississippi River," Trans., ASCE, Vol. XL, pp. 215-310.

Odd, N., 1965, "Maintenance Dredging of Cohesive Sediments in Tidal Estuaries," Unpublished Manuscript, University of California, Berkeley.

Odom, B. W., 1966, "River Regulation Works on the Apalachicola River," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 4, Chap. VII.

O'Neal, G. and Sceva, J., 1971, "The Effects of Dredging on Water Quality in the Northwest," Environmental Protection Agency, Office of Water Programs, Region X, Seattle, Washington.

Oregon State University, 1972, "Integrated Approaches to Examine Dredging Needs and Future Consequences," Proc., Pacific Chapter World Dredging Association.

Permanent International Association of Navigation Congresses (PIANC), 1960, "Report on Dredging of Navigable Waterways and Inland Ports."

Petersen, M. S., 1964, "Hydraulic Aspects of Arkansas River Stabilization," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 2, Chap. V.

Rhodes, N. C., 1972, "Dredging Operations and the Environment," Proc., World Dredging Conference 1972, (WODCON), pp. 719-727.

Rubey, W. W., 1952, "Geology and Mineral Resources of the Hardin and Brussels Quadrangles (In Illinois)," USGS Prof. Paper 218, 179 p.

Schmidt, F. J., 1972, "The Dustpan Dredge--An American Development and Its Future Possibilities," Proc., World Dredging Conference 1972, (WODCON), pp. 49-92.

Schumm, S. A., 1960, "The Shape of Alluvial Channels in Relation to Sediment Type," USGS Prof. Paper 352-B, 30 p.

Scott, H. A., 1972, "Design Versus Ecology in St. Johns and Indian Rivers," Jour. of Waterways, Harbors and Coastal Engineering Division, ASCE, Vol. 98, No. WW4, November, pp. 425-432.

Senour, C., 1947, "New Project for Stabilizing and Deepening Lower Mississippi River," Trans., ASCE, Vol. 112, Paper 2299, pp. 277-290.

Simon, F. L., 1920, "Dredging Engineering," McGraw-Hill Book Co., Inc., New York.

Simons, D. B., 1973, "Possible Effects of Dredging Gravel from the Mississippi River," Communication dated 19 February 1973, Appendix A to Winkley and Harris, 1973.

Simons, D. B., Schumm, S. A. and Stevens, M. A., 1974, "Geomorphology of the Middle Mississippi," Colorado State University, Technical Report for the Waterways Experiment Station, Vicksburg, Mississippi

Simons, D. B., Schumm, S. A., Stevens, M. A., Chen, Y. H. and Lagasse, P. F., 1975, "A Geomorphic Study of Pools 24, 25, and 26 in the Upper Mississippi and Lower Illinois Rivers," Colorado State University, Technical Report for the Waterways Experiment Station, Vicksburg, Mississippi.

Smith, A. B., 1963, "Channel Sedimentation and Dredging Problems, Mississippi River and Louisiana Gulf Coast Access Channels," Proc. of the Federal Inter-Agency Sedimentation Conference, Jackson, Mississippi, pp. 618-625.

Solomon, R. C., Johnson, J. H., Bingham, R. C., Colbert, B. K. and Hughes, G. W., 1974, "Physical, Biological, and Chemical Inventory and Analysis of Selected Dredged and Disposal Sites-Middle Mississippi River," Working Draft, Waterways Experiment Station, Miscellaneous Paper.

Somervell, B., 1932, "Report to the President of the Mississippi River Commission on Dredging in the Mississippi River between Cairo and the Mouth of the Arkansas River for the Improvement of Navigation," with Appendix B, "History of Dredging on the Mississippi River Below Cairo, Illinois," U.S. Engineer Office, Memphis, Tennessee, available from Mississippi River Commission Library, Vicksburg, Mississippi.

Teal, J. and Teal, M., 1966, "Life and Death of a Salt Marsh," Audubon/Ballantine Book, Intext, New York.

Texas A & M, 1969, "Proc. of the Second Dredging Seminar," Sea Grant Publ. No. TAMU-SG-70-113, Coastal and Ocean Engineering Division Report No. 126-CDS.

Tiffany, J. B., 1963, "Review of Research on Channel Stabilization of the Mississippi River 1931-1962," Technical Report No. 2, Corps of Engineers, Committee on Channel Stabilization.

Tiffany, J. B., 1963, "Recommended Research on Channel Stabilization," Technical Report No. 1, Corps of Engineers, Committee on Channel Stabilization, Vol. 1, Chap. VII.

Turner, T. M., 1970, "The Basic Dredge Laws," Proc., World Dredging Conference 1970, (WODCON), pp. 411-424.

Turner, T. M. and Fairweather, V., 1974, "Dredging and the Environment: The Plus Side," Civil Engineering, ASCE, Vol. 44, No. 10, October, p. 62-65.

United States District Court for the Western District of Wisconsin, 1973, "Opinion and Order 73-C-183," July 10, 1973.

Upper Mississippi River Basin Coordinating Committee, 1972, "Upper Mississippi River Comprehensive Basin Study (UMRCBS)," Vols. I, II, III, V, and VI.

Upper Mississippi River Conservation Committee--Fish Technical Section, 1970, "1969 Upper Mississippi River Dredge Spoil Survey from Hastings, Minnesota to Cairo, Illinois," Missouri Dept. of Conservation.

Upper Mississippi River Conservation Committee--Fish Technical Section, 1971, "Upper Mississippi River Habitat Classification Survey--Hastings, Minnesota to Alton, Illinois," Minnesota Dept. of Natural Resources.

Vinje, J. J., 1968, "Siltation in Dredged Trenches," Proc., World Dredging Conference 1968, (WODCON), pp. 851-891.

Vittor, B. A., 1972, "An Experimental Approach to the Use of Dredging Spoil for the Construction of Salt Marsh Islands," Proc., World Dredging Conference 1972, (WODCON), pp. 599-610.

Wilcock, D. N., 1971, "Investigation into the Relations between Bedload Transport and Channel Shape," Bulletin, Geologic Society of America, Vol. 82, No. 8, August, pp. 2159-2176.

Windom, H. L., 1972, "Environmental Aspects of Dredging in Estuaries," Jour. of Waterways, Harbors and Coastal Engineering Division, ASCE, Vol. 98, No. WW4, November, pp. 475-488.

Winkley, B. R., 1971, "Practical Aspects of River Regulation and Control," River Mechanics, H. W. Shen, ed., Vol. 1, Chap. 19, Water Resources Publ., Fort Collins, Colorado.

Winkley, B. R., 1973, "Metamorphosis of a River--A Comparison of the Mississippi River before and after Cutoffs," Water Resources Research Institute, Mississippi State University.

Winkley, B. R. and Harris, P. C., 1973, "Preliminary Investigation of the Effects of Gravel Mining in the Mississippi River," Corps of Engineers, Vicksburg District, Potamology Section, River Stabilization Branch.

Chapter 8

DATA NEEDS AND DATA SOURCES

Barnes, H. H., Jr., 1967, "Roughness Characteristics of Natural Channels," USGS Water Supply Paper 1849, 213 p.

Chen, Y. H., 1975, "Data Needs for Geomorphic and Hydraulic Study of Rivers," Unpublished draft, Colorado State University, Fort Collins, Colorado.

Dalrymple, T., 1960, "Flood-Frequency Analysis," USGS Water Supply Paper 1543-A, 80 p.

Karaki, S., Mahmood, K., Richardson, E. V., Simons, D. B. and Stevens, M. A., 1974, "Highways in the River Environment, Hydraulic and Environmental Design Considerations," Report prepared for the Federal Highway Administration, Colorado State University, Fort Collins, Colorado.

Potter, W. D., 1961, "Hydrology of a Highway Stream Crossing," Office of Engineering and Operations, U.S. Bureau of Public Roads.

Simons, D. B. and Li, R. M., 1975, "Model Formulation and Data Needs for Mathematical Modeling of Watershed Response," Unpublished draft, Colorado State University, Fort Collins, Colorado.

Task Force on Hydrology and Hydraulics, 1973, "Guidelines for Hydrology," AASHO Operating Subcommittee on Roadway Design, Vol. II of Highway Drainage Guidelines.

U.S. Water Resources Council, 1967, "A Uniform Technique for Determining Flood Flow Frequencies."

APPENDIX B

DEVELOPMENT OF THE BASIC EQUATIONS OF OPEN CHANNEL FLOW

B.1 Conservation of Mass

Consider a short reach of river shown in Figure B-1 as a control volume. The boundaries of the control volume are the upstream cross section (designated Section 1), the downstream cross section (designated Section 2), the free surface of the water between Sections 1 and 2, and the interface between the water and the banks and bed. Here V is the flow velocity, ρ is the mass density, dA_1 represents an increment of cross-sectional area, and dx represents an increment along the path of flow.

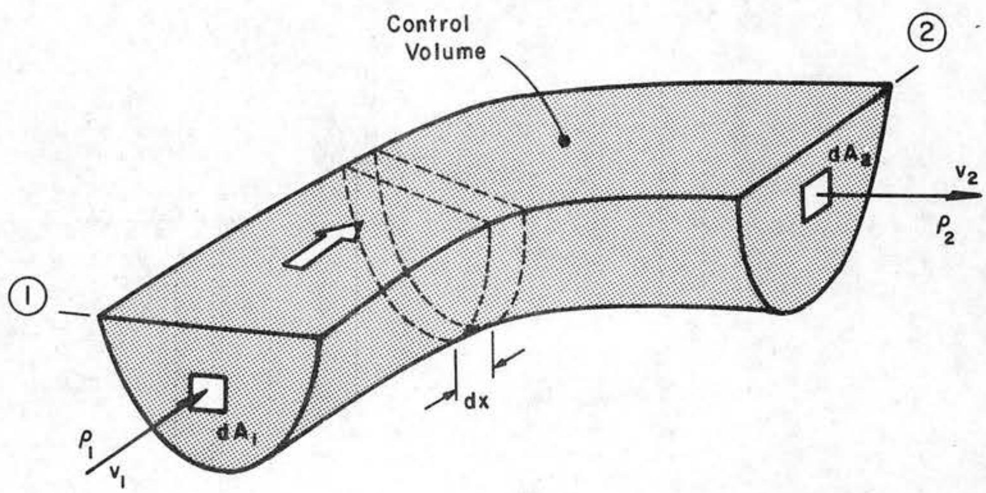


Figure B-1 A River Reach as a Control Volume.

The statement of the conservation of mass for this control volume is

Mass flux out of the control volume	-	Mass flux into the control volume	+	Time rate of change in mass in the control volume	=	0

Mass can enter or leave the control volume through any or all of the control volume surfaces. Rainfall would contribute mass through the surface of the control volume and seepage passes through the interface between the water and the banks and bed. In the absence of rainfall evaporation, seepage, and other lateral mass fluxes, mass enters the control volume at Section 1 and leaves at Section 2.

At Section 2, the mass flux out of the control volume through the differential area dA_2 is $\rho_2 v_2\ dA_2$. The values of ρ_2 and v_2 can vary from position to position across the width and throughout the depth of flow at Section 2. The total mass flux out of the control volume at Section 2 is the sum of all $\rho_2 v_2\ dA_2$ through the differential areas that make up the cross section area A_2, and may be written as

$$\sum_{A_2} \rho_2 v_2\ dA$$

Therefore

$$\begin{array}{l}\text{Mass flux}\\ \text{out of the}\\ \text{control volume}\end{array} = \sum_{A_2} \rho_2 v_2\ dA_2$$

Similarly

$$\begin{array}{l}\text{Mass flux}\\ \text{into the}\\ \text{control volume}\end{array} = \sum_{A_1} \rho_1 v_1\ dA_1$$

The amount of mass inside a differential volume $d\forall$ inside the control volume is

$$\rho\ d\forall$$

and the total mass inside the control volume $\forall$ is then the sum of the mass inside or

$$\begin{array}{l}\text{Mass inside}\\ \text{the}\\ \text{control volume}\end{array} = \sum_{\forall} \rho\ d\forall$$

The statement of conservation of mass for the control volume includes the time rate of change in mass in the control volume. In mathematical notation, this rate of change is

$$\begin{array}{l}\text{Time rate of change}\\ \text{in mass in the}\\ \text{control volume}\end{array} = \frac{\partial}{\partial t} \{\sum_{\forall} \rho\ d\forall\}$$

For the reach of river, the statement of the conservation of mass becomes

$$\sum_{A_2} \rho_2 v_2 \, dA_2 - \sum_{A_1} \rho_1 v_1 \, dA_1 + \frac{\partial}{\partial t} \{\sum_{\forall} \rho \, d\forall\} = 0 \qquad (B.1)$$

It is often convenient to work with average conditions at a cross section so we define an average velocity V such that

$$V = \frac{1}{A} \sum_{A} v \, dA$$

or in integral form,

$$V = \frac{1}{A} \int_{A} v \, dA$$

The velocity V is the <u>average velocity</u> at the cross section.

If the density of the fluid does not change from position to position in a cross section or in the reach, $\rho_1 = \rho_2 = \rho$. When the flow is steady

$$\frac{\partial}{\partial t} \{\sum_{\forall} \rho \, d\forall\} = 0$$

and Equation (B.1) reduces to the statement that inflow equals outflow or

$$\rho V_2 A_2 - \rho V_1 A_1 = 0$$

That is,

$$V_1 A_1 = V_2 A_2 = Q \qquad (B.2)$$

where Q is the volume flow rate or the discharge.

<u>Equation (B.2) is the familiar form of the conservation of mass equation or continuity equation for river flows</u> (see Equation 2.6). It is applicable when the fluid density is constant, the flow is steady, and there are no lateral inflows or seepage. For unsteady flow with lateral flows, the continuity equation (B.1) can be expressed in the form of Equation (2.90).

B.2 Conservation of Linear Momentum

The curved reach of river shown in Figure B-1 is rather complex to analyze in terms of Newton's Second Law because of the curvature in the flow. Therefore, as a starting point, the differential length of reach dx is isolated as a control volume.

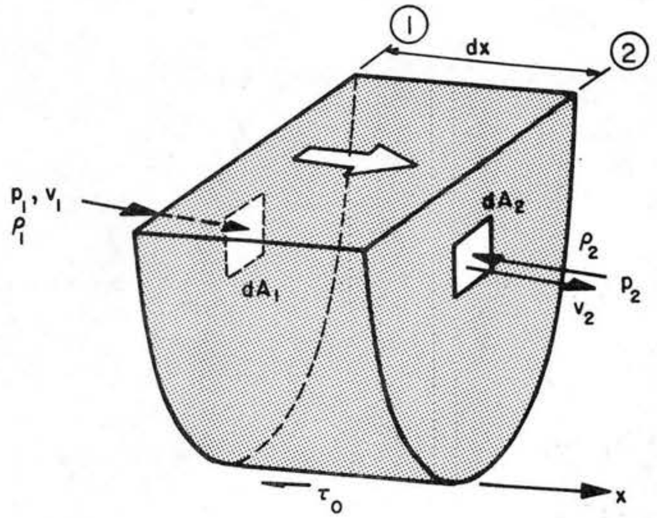


Figure B-2 The Control Volume for Conservation of Linear Momentum.

For the control volume shown in Figure B-2, the statement of conservation of linear momentum is

	Flux of momentum out of the control volume	-	Flux of momentum into the control volume	
+	Time rate of change of momentum in the control volume	=	Sum of the forces acting on the fluid in the control volume	

<u>The terms in the statement are vectors so we must be concerned with direction as well as magnitude.</u>

Consider the conservation of momentum in the direction of flow (the x-direction in Figure B-2). At the outflow section (Section 2), the flux of momentum out of the control volume through the differential area dA_2 is

$$\rho_2 v_2 \, dA_2 \, v_2$$

Here $\rho_2 v_2 \, dA_2$ is the mass flux (mass per unit of time) and $\rho_2 v_2 \, dA_2 \, v_2$ is the momentum flux through the area dA_2. The total momentum flux out through Section 2 is

$$\text{Flux of momentum out of the control volume} = \sum_{A_2} \rho_2 v_2 \, dA_2 \, v_2$$

Similarly, at the inflow section (Section 1)

$$\text{Flux of momentum into the control volume} = \sum_{A_1} \rho_1 v_1 \, dA_1 \, v_1$$

The amount of momentum in the control volume is

$$\sum_{\forall} \rho v \, d\forall$$

so

$$\text{Time rate of change of momentum in the control volume} = \frac{\partial}{\partial t} \{\sum_{\forall} \rho v \, d\forall\}$$

At the upstream section, the force acting on the differential area dA_1 of the control volume is

$$p_1 dA_1$$

where p_1 is the pressure from the upstream fluid on the differential area.

The total force in the x-direction at Section 1 is

$$\sum_{A_1} p_1 \, dA_1$$

Similarly, at Section 2, the total force is

$$- \sum_{A_2} p_2 \, dA_2$$

There is a fluid shear stress acting along the interface between the water and the bed and banks. The shear on the control volume is in a direction opposite to the direction of flow and results in a force

$$-\tau_o P \, dx$$

where τ_o is the average shear stress on the interface area, P is the average wetted perimeter and dx is the length of the control volume. The term $P \, dx$ is the interface area.

If the x-direction is normal to the direction of gravity, the statement of conservation of momentum in the x-direction for the control volume is

$$\sum_{A_2} \rho_2 v_2 v_2 \, dA_2 - \sum_{A_1} \rho_1 v_1 v_1 \, dA_1 + \frac{\partial}{\partial t} \{\sum_{\forall} \rho v \, d\forall\}$$

$$= \sum_{A_1} p_1 dA_1 - \sum_{A_2} p_2 dA_2 - \tau_o P \, dx \qquad \text{(B.3)}$$

In the limit, the summations can be replaced with integrals so that Equation (B.3) becomes

$$\int_{A_2} \rho_2 \, v_2^2 \, dA_2 - \int_{A_2} \rho_1 \, v_1^2 \, dA_1 + \frac{\partial}{\partial t} \{\int_{\forall} \rho v \, d\forall\}$$

$$= \int_{A_1} p_1 \, dA_1 - \int_{A_2} p_2 \, dA_2 - \tau_o P \, dx \qquad \text{(B.4)}$$

which is the integral form of the momentum equation.

Again, as with the conservation of mass equation, it is convenient to use average velocities instead of point velocities. We define a momentum coefficient β so that when average velocities are used instead of point velocities, the correct momentum flux is considered. The momentum coefficient is

$$\beta = \frac{1}{\rho V^2 A} \int_A \rho v^2 \, dA \qquad \text{(B.5)}$$

which reduces to

$$\beta = \frac{1}{V^2 A} \int_A v^2 \, dA \tag{B.6}$$

if there is no variation in fluid density at a cross section. By assuming $\rho_1 = \rho_2 = \rho$, Equation (B.4) is reduced to

$$\rho\beta_2 V_2^2 A_2 - \rho\beta_1 V_1^2 A_1 + \rho \frac{\partial}{\partial t} \{\int_{\forall} v \, d\forall\}$$

$$= \int_{A_1} p_1 \, dA_1 - \int_{A_2} p_2 \, dA_2 - \tau_o P \, dx \tag{B.7}$$

This equation can be formulated in partial differential form to yield Equation (2.91). If the flow is steady

$$\frac{\partial}{\partial t} \{\int_{\forall} v \, d\forall\} = 0$$

The pressure force and shear force terms on the right-hand side of Equation (B.4) are usually abbreviated as $\sum F_x$ or

$$\sum F_x = \int_{A_1} p_1 \, dA_1 - \int_{A_2} p_2 \, dA_2 - \tau_o P \, dx$$

Then, for steady flow of constant density fluid, the conservation of momentum equation becomes

$$\rho\beta_2 V_2^2 A_2 - \rho\beta_1 V_1^2 A_1 = \sum F_x \tag{B.8}$$

For steady flow with constant density, the conservation of mass equation (B.2) was

$$V_1 A_1 = V_2 A_2 = Q$$

With this expression, <u>the steady flow conservation of linear momentum equation takes on the familiar form</u> (see Equation 2.12)

$$\rho Q \, (\beta_2 V_2 - \beta_1 V_1) = \sum F_x \tag{B.9}$$

B.3 Conservation of Energy

The First Law of Thermodynamics can be written

$$\dot{Q} - \dot{W} = \frac{dE}{dt} \tag{B.10}$$

where $\dot{Q}$ = the rate at which heat is added to a fluid system

$\dot{W}$ = the rate at which a fluid system does work on its surroundings

E = the energy of the system

Then dE/dt is the rate of change of energy in the system.

The statement of conservation of energy for a control volume is then

$$\begin{pmatrix}\text{Flux of energy}\\ \text{out of the}\\ \text{control volume}\end{pmatrix} - \begin{pmatrix}\text{Flux of energy}\\ \text{into the}\\ \text{control volume}\end{pmatrix} + \begin{pmatrix}\text{Time rate of}\\ \text{change of energy}\\ \text{in the control volume}\end{pmatrix} = \dot{Q} - \dot{W}$$

The choice of a control volume is arbitrary. Because of the complexities resulting from having to integrate over the cross-sectional area, a control volume which includes the entire cross section of the river is inconvenient. Therefore, the control volume is reduced to the size of a streamtube connecting dA_1 and dA_2 as shown in Figure B-3. The streamtube is bounded by streamlines through which no mass or momentum enters.

For steady constant density flow in the streamtube

$$\begin{pmatrix}\text{Flux of energy}\\ \text{out of the}\\ \text{control volume}\end{pmatrix} = \rho_2 e_2 \, dA_2 \, v_2$$

and

$$\begin{pmatrix}\text{Flux of energy}\\ \text{into the}\\ \text{control volume}\end{pmatrix} = \rho_1 e_1 \, dA_1 \, v_1$$

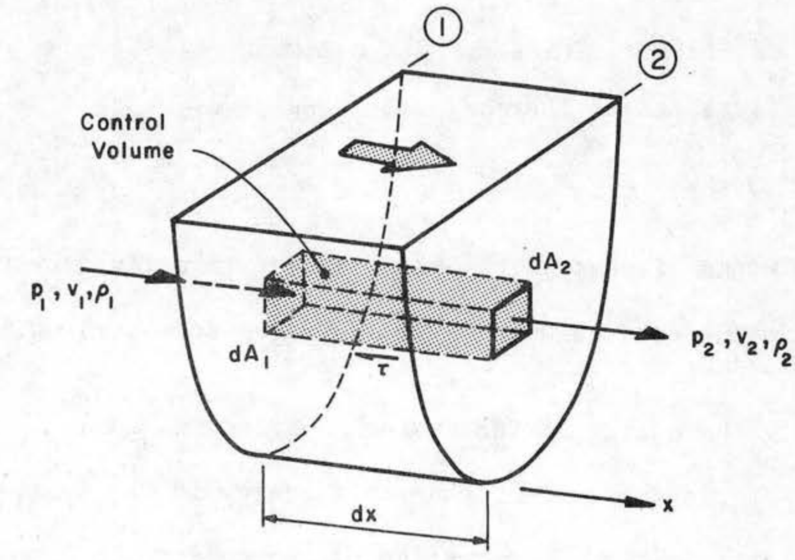


Figure B-3 The Streamtube as a Control Volume.

Here e is the energy per unit of mass. Accordingly, the total energy in a control volume of size $\forall$ is

$$E = \int_{\forall} \rho e \, d\forall \tag{B.11}$$

If the flow is assumed steady

Time rate of
change of energy = 0
in the control volume

Unless one is concerned with thermal pollution, evaporation losses, or problems concerning the formation of ice in rivers, the rate at which heat is added to the control volume can be neglected; that is,

$$\dot{Q} \simeq 0 \tag{B.12}$$

The work done by the fluid in the control volume on its surroundings can be in the form of pressure work W_p, shear work W_τ, or shaft work (mechanical work) W_s. For the streamtube shown in Figure B-3, no shaft work is involved.

The rate at which the fluid pressure does work on the control volume boundary dA_1 in Figure B-3 is

$$-p_1\ dA_1\ v_1$$

and on boundary dA_2, the rate of doing pressure work is

$$p_2\ dA_2\ v_2$$

At the other boundaries of the streamtube, there is no pressure work because there is no fluid motion normal to the boundary. Hence, for the streamtube

$$\dot{W}_p = p_2\ dA_2\ v_2 - p_1\ dA_1\ v_1 \tag{B.13}$$

Along the interior boundaries of the streamtube there is a shear stress resulting from the possible condition that the fluid velocity inside the streamtube may not be the same as the velocity of the fluid surrounding the streamtube. The rate at which the fluid in the streamtube does shear work on the control volume is

$$\dot{W}_\tau = \tau P\ dx\ v \tag{B.14}$$

where τ is the average shear stress on the streamtube boundary, P is the average perimeter of the streamtube, dx is the length of the streamtube and v is the fluid velocity at the streamtube boundary. The product $P\ dx$ is the surface of the streamtube subjected to shear stresses.

Then for steady flow in the streamtube, the statement of the conservation of energy in the streamtube shown in Figure B-3 is

$$\rho_2 e_2 v_2\ dA_2 - \rho_1 e_1 v_1\ dA_1 = p_1 v_1\ dA_1 - p_2 v_2\ dA_2 - \tau P v\ dx \tag{B.15}$$

If the density is constant in the streamtube, the conservation of mass for the streamtube is (according to Section B.1)

$$v_2\ dA_2 = v_1\ dA_1 = dQ$$

and

$$\rho_2 = \rho_1 = \rho. \tag{B.16}$$

Now Equation (B.15) reduces to

$$(\rho e_1 + p_1)dQ - (\rho e_2 + p_2)dQ = \tau Pv\ dx \tag{B.17}$$

The energy per unit mass e is the sum of the internal, kinetic and potential energies or

$$e = u + \frac{v^2}{2} + gz \tag{B.18}$$

where u = the internal energy associated with the fluid temperature

v = the velocity of the mass of fluid

g = the acceleration due to gravity

z = the elevation above some arbitrary reference level.

This expression for e is substituted in Equation (B.17) to yield

$$u_1 + \frac{v_1^2}{2} + gz_1 + \frac{p_1}{\rho} = u_2 + \frac{v_2^2}{2} + gz_2 + \frac{p_2}{\rho} + \frac{\tau Pvdx}{\rho dQ} \tag{B.19}$$

By dividing through by g and calling the term

$$\frac{u_2 - u_1}{g} + \frac{\tau Pvdx}{\rho g dQ}$$

the head loss h_ℓ, the energy equation for the streamtube becomes

$$\frac{v_1^2}{2g} + \frac{p_1}{\gamma} + z_1 = \frac{v_2^2}{2g} + \frac{p_2}{\gamma} + z_2 + h_\ell \tag{B.20}$$

<u>If there is no shear stress on the streamtube boundary and if there is no change in internal energy ($u_1 = u_2$), the energy equation reduces to</u>

$$\frac{v_1^2}{2g} + \frac{p_1}{\gamma} + z_1 = \frac{v_2^2}{2g} + \frac{p_2}{\gamma} + z_2 \tag{B.21}$$

which is the <u>Bernoulli Equation</u>.

Generally, there is not sufficient information available to do a differential streamtube analysis of a reach of river, so appropriate changes must be made in the energy equation. A reach of river such as that shown in Figure B-1 can be pictured as a bundle of streamtubes. The statement of the conservation of energy for a streamtube is Equation (B.20) which can be rewritten

$$\left(\frac{v_1^2}{2g} + \frac{p_1}{\gamma} + z_1\right) v_1 \, dA_1 = \left(\frac{v_2^2}{2g} + \frac{p_2}{\gamma} + z_2\right) v_2 \, dA_2 + h_\ell \, vdA \qquad \text{(B.22)}$$

because $v_1 \, dA_1 = v_2 \, dA_2 = vdA$

for the streamtube. By summing the energies in all the streamtubes making up the reach of river, we can write

$$\frac{\alpha_1 V_1^2}{2g} + \frac{\overline{p}_1}{\gamma} + \overline{z}_1 = \frac{\alpha_2 V_2^2}{2g} + \frac{\overline{p}_2}{\gamma} + \overline{z}_2 + H_L \qquad \text{(B.23)}$$

Equation (B.23) is the common form of the energy equation used in open channel flow. It is derived from Equation (B.22) by integrating Equation (B.22) over the cross section area; that is

$$\int_A \left(\frac{v^2}{2g} + \frac{p}{\gamma} + z\right) vdA = \left\{\frac{\alpha V^2}{2g} + \frac{\overline{p}}{\gamma} + \overline{z}\right\} Q \qquad \text{(B.24)}$$

where α is the <u>kinetic energy correction factor</u> defined by the expression

$$\alpha = \frac{1}{V^3 A} \int_A v^3 \, dA \qquad \text{(B.25)}$$

to allow the use of average velocity V rather than point velocity v. The average pressure over the cross section is $\overline{p}$, defined as

$$\overline{p} = \frac{1}{VA} \int_A p \, vdA \qquad \text{(B.26)}$$

The term $\overline{z}$ is the average elevation of the cross section defined by the expression

$$\bar{z} = \frac{1}{VA} \int_A z \, vdA \tag{B.27}$$

and Q is the volume flow rate or the discharge. By definition

$$Q = \int_A vdA \tag{B.28}$$

Also

$$H_L = \frac{1}{VA} \int_A h_\ell \, vdA \tag{B.29}$$

In summary, the <u>expression for conservation of energy for steady flow in a reach of river is written</u> (see Equation 2.18)

$$\frac{\alpha_1 V_1^2}{2g} + \frac{\bar{p}_1}{\gamma} + \bar{z}_1 = \frac{\alpha_2 V_2^2}{2g} + \frac{\bar{p}_2}{\gamma} + \bar{z}_2 + H_L \tag{B.30}$$

The tendency in river work is to neglect the energy correction factor even though its value may be as large as 1.5. Usually it is assumed that the pressure is hydrostatic and the average elevation head $\bar{z}$ is at the centroid of the cross-sectional area. However, it should be kept in mind that Equations (B.25, B.26, and B.27) are the correct definitions of the terms in the energy equation.

APPENDIX C

GLOSSARY

Abrasion. Act or process of wearing down by friction, or the resulting effects. Moving debris, whether it be in a stream, sea, ice and wind.

Acre-Foot. A unit for measuring the volume of water, is equal to the quantity of water required to cover 1 acre to a depth of 1 foot and is equal to 43,560 cubic feet or 325,851 gallons. The term is commonly used in measuring volumes of water used or stored.

Aggradation. Process of raising a land surface by the deposition of sediment.

Alluvial Channel. A channel whose bed is composed of noncohesive sediment that has been or can be transported by the flow.

Alluvial Fan. Alluvial deposit of a stream where it issues from a gorge upon an open plain.

Alluvial Plain. Plain formed by the deposition of alluvial material eroded from area of higher elevation.

Alluvium (Alluvial Deposit). Clay, silt, sand, gravel, pebble or other detrital material deposited by water.

Anabranch. A diverging branch of a river which reenters the main stream.

Antidunes. Bed forms of curved symmetrically shaped sand waves that may move upstream, remain stationary, or move downstream. They occur in trains that are inphase with and strongly interact with gravity water-surface waves. The water-surface waves have larger amplitudes than the coupled sand waves. At large Froude number, the waves generally move upstream and grow until they become unstable and break like surf (breaking antidunes). The agitation accompanying the breaking obliterates the antidunes, and the process of antidune initiation and growth is repeated. At small Froude numbers the antidunes generally remain stationary and increase and then decrease in amplitude without breaking (standing waves).

Apron. An adjunct to a dam or other structure, consisting of a surface protection against erosion.

Backwater. Water backed up or retarded in its course as compared with its normal or natural condition of flow. In stream gaging, a rise in stage produced by a temporary obstruction such as ice or weeds, or by the flooding of the stream below.

Backwater Curve. Longitudinal profile of the water surface in a stream where the water surface is raised above its normal level by a natural or artificial obstruction.

Bank. The margins of a channel. Banks are called right or left as viewed facing in the direction of the flow.

Bankfull Stage. Stage at which a stream first overflows its natural bank.

Barrage. A dam.

Bars. Bed forms having lengths of the same order as the channel width or greater, and heights comparable to the mean depth of the generating flow.

Bars, Alternate. Bars occured in straighter reaches of channels and tend to be distributed periodically along the reach, with consecutive bars on opposite sides of the channel. Their lateral extent is significantly less than the channel width.

Bars, Middle (or Transverse). Bars occurred in straight channels and occupied the full channel width.

Bars, Point. Bars occurred adjacent to the convex bank of channel bends.

Bars, Tributary. Bars occurred immediately downstream from points of lateral inflow into a channel.

Bed (Streambed). The bottom of a water course.

Bed Configuration. A complex of bed forms covering the bed of an alluvial stream.

Bed Form. A generic term used to denote any irregularity produced on the bed of an alluvial channel by flowing water and sediment.

Bed Layer. A flow layer, several grain diameter thick (usually taken as two grain diameter thick) immediately above the bed.

Bed Load. That part of the total sediment load that moves by rolling or sliding along the bed. The term "bed load" may be used to designate either coarse material moving on or near the bed, or material collected in or computed from samples collected in a bed load sampler or trap. In other words, load which is not sampled by a suspension load sampler.

Bed-Load Discharge. The quantity of bed load passing any cross section of a stream in a unit of time.

Bed-Load Discharge Sampler. A device to measure the discharge of bed load over part or all of the stream width.

Bed Material. The material of which a streambed is composed.

Bed-Material Discharge. Sediment discharge that consists of particles large enough to be found in appreciable quantities in the streambed.

Bed-Material Load. That part of the total sediment load which is compose of grain sizes represented in the bed--equal to the transport capacity of the flow.

Benthic Community. A group of plants or animals living in or on the streambed.

Braiding of River Channels. Successive division and rejoining (of riverflow) with accompanying islands is the important characteristic denoted by the synonymous terms, braided or anastomosing stream. (Leopold and Wolman, 1957, p. 40.) A braided stream is composed of anabranches.

Breaking Antidune. Curved symmetrically shaped waves on the water surface and on the channel bottom that build up with time and then break like surf.

Canal. An open conduit for the conveyance of water; distinguished from a ditch or lateral by its larger size; usually excavated in natural ground.

Capacity. The ability of a stream current to transport in terms of quantity.

Capture. Diversion of the flow of water in the upper part of a stream by the headward growth of another stream.

Channel. (1) Deepest portion of a river bed, in which the main current flows. (2) Natural or artificial, clearly distinguished, waterway which periodically or continuously contains moving water, or which forms a connecting link between two bodies of water.

Channel, Backwater. Side channels which do not carry appreciable flows even at high stage.

Channel, Side. The smaller channels in a reach of river where islands divide the reach into two or more channels. The larger is referred to as the main or thalweg channel.

Channel, Stable. Channel in which accretion balances scour on the average.

Channel, Straight. A channel having its sinuosity less than 1.5.

Chute. Natural or artificial steep-sloped reach of an open channel.

Chute and Pools. The flow phenomenon and bed configuration accompanying flows that occur at steep slopes and large bed-material discharges. The flow occurs at slopes steeper than for antidunes and consists of a series of pools in which the flow is tranquil, connected by steep chutes where the flow is rapid. A hydraulic jump forms at the downstream end of each chute where it enters the pool. The bed configuration consists of triangle-shaped elements with a steep upstream slope, a flat, almost horizontal back, and a gently downstream slope. The chutes and pools move slowly upstream.

Clay. Sediment finer than 0.004 mm (millimeters) regardless of mineralogical composition.

Competency. The ability of currents to transport, in terms of dimensions of particles.

Confluence. Joining, or the place of junction, of two or more streams.

Contact Load. Sediment particles that roll or slide along in almost continuous contact with the streambed (often used synonomously with bed load).

Control. A natural constriction of the channel, a long reach of the channel, a stretch of rapids, or an artificial structure downstream from a gaging station that determines the stage-discharge relation at the gage.

Critical Flow. Flow conditions at which the discharge is a maximum for a given specific energy, or at which the specific energy is minimum for a given discharge.

Crossing and Pool. A series of shoals (crossings or bars) and deep (pools) sequence exhibited in rivers.

Crossover. Relatively short and shallow length of a river between bends.

Cross Section (of a Stream). Section of the stream at right angle to the main (average) direction of flow.

Cubic Feet per Second. A unit expressing rates of discharge. One cubic foot per second is equal to the discharge of a stream of rectangular cross section, 1 ft wide and 1 ft deep, flowing water an average velocity of 1 ft per second.

Cusec. This abbreviation for cubic foot per second, common in the British Commonwealth countries (except Canada), is not used by the U.S. Geological Survey; instead, cfs is used.

Cut-off (Cutoff). Direct channel, either natural or artificial, connecting two points on a stream, thus shortening the length of the channel and increasing its slope.

Degradation. Disintegration and wearing down of the surface of rocks, cliffs, strata, streambeds, etc. by atmospheric and aqueous action.

Delta. Alluvial deposit at the mouth of a river and the geographical and geomorphological unit which results from it.

Density, Water-Sediment Mixture. The bulk density which is the mass per unit volume including both water and sediment.

Depth-Integrated Sample. A water-sediment mixture that is accumulated continuously in a sampler that moves vertically at an approximately constant transit rate between the surface and a point a few inches above the bed of a stream, and that admits the mixture at a velocity about equal to the instantaneous stream velocity at each point in the vertical. Because the sampler intake is a few inches above the sampler bottom, there is an unsampled zone a few inches deep just above the bed of the stream.

Detritus. Any loose material that results directly from rock disintegration, especially when composed of rock fragments--contrasted with soil. In the sediment field detritus has generally been used to designate the coarser material moved or deposited.

Discharge. In its simplest concept discharge means outflow; therefore, the use of this term is not restricted as to course or location and it can be applied to describe the flow of water from a pipe or from a drainage basin. If the discharge occurs in some course or channel, it is correct to speak of the discharge of a canal or of a river. It is also correct to speak of the discharge of a canal or stream into a lake, a stream, or an ocean.

Discharge-Weighted Concentration. The dry weight of sediment in a unit volume of stream discharge, or the ratio of the discharge of dry weight of sediment to the discharge by weight of water sediment mixture.

Disposal, On Land. Disposal of dredged material on land at locations where the materials are not subjected to the influence of water stage fluctuation.

Disposal, Open Water. Disposal of dredged material 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.

Disposal, Thalweg. Disposal of dredged material in the main channel.

Diversion. The taking of water from a stream or other body of water into a canal, pipe, or other conduit.

Diversion Dam. A dam built for the purpose of diverting part or all the water from a stream into a different course.

Drainage Basin. A part of the surface of the earth that is occupied by a drainage system, which consists of a surface stream or a body of impounded surface water together with all tributary surface streams and bodies of impounded surface water.

Drainage Divide. The rim of a drainage basin.

Dredging. 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.

Dunes. Large bed forms having triangular profiles, a gentle upstream slope, and a steep downstream slope. They form in tranquil flow and, thus, are out of phase with any water-surface disturbance that they may produce. They travel slowly downstream as sand is moved across their comparatively gentle, upstream slopes and deposited on their steeper, downstream slopes. The downstream slopes are approximately equal to the angle of repose of the bed material. Dunes are smaller than sand bars but larger than ripples. They generally form at higher velocities and larger sediment discharges than do ripples, but at lower velocities and smaller sediment discharges than do antidunes. However, ripples form on the upstream slopes of dunes at lower velocities.

Eutrophication. Process by which waters become more eutrophic (richer in dissolved nutrients required for the growth of aquatic plants such as algae) either as a natural phase in the maturation of a body of water or artificially (as by fertilization and pollution).

Evapotranspiration. Water withdrawn from a land area by evaporation from water surfaces and moist soil and plant transpiration.

Fall Diameter or Standard Fall Diameter. The diameter of a sphere that has a specific gravity of 2.65 and the same terminal uniform settling velocity as the particle (any specific gravity) when each is allowed to settle alone in quiescent distilled water of infinite extent and at a temperature of 24°C.

Fall Velocity. Average terminal settling velocity of a particle falling alone in quiescent, distilled water of infinite extent.

Fine Sediment. That part of the sediment discharge that consists of sediment so fine that it is about uniformly distributed in the vertical and is only an inappreciable fraction of the sediment in the streambed (referred to by some writers as washload). Its upper size limit at a particular time and cross section is a function of the flow as well as of the sediment particles.

Flood. An overflow or inundation that comes from a river or other body of water (Barros, 1948) and causes or threatens damage. Any relatively high streamflow overtopping the natural or artificial banks in any reach of a stream (Leopold and Maddock, 1954, pp. 249-251).

Flood-Frequency Curve. 1. A graph showing the number of times per year on the average, plotted as abscissa, that floods of magnitude, indicated by the ordinate, are equaled or exceeded. 2. A similar graph but with recurrence intervals of floods plotted as abscissa.

Flood Peak. The highest value of the stage or discharge attained by a flood; thus, peak stage or peak discharge. Flood crest has nearly the same meaning, but since it connotes the top of the flood wave, it is properly used only in referring to stage--thus, crest stage, but not crest discharge.

Flood Plain. A strip of relatively smooth land bordering a stream, built of sediment carried by the stream and dropped in the slack water beyond the influence of the swiftest current. It is called a living flood plain if it is overflowed in times of highwater; but a fossil flood plain if it is beyond the reach of the highest flood.

Flood Routing. The process of determining progressively the timing and shape of a flood wave at successive points along a river.

Flood Stage. The stage at which overflow of the natural banks of a stream begins to cause damage in the reach in which the elevation is measured.

Flood Wave. A distinct rise in stage culminating in a crest and followed by recession to lower stages.

Floodway. A part of the flood plain, otherwise leveed, reserved for emergency diversion of water during floods. A part of the flood plain which, to facilitate the passage of floodwater, is kept clear of encumbrances.

Flow-Duration Curve. A cumulative frequency curve that shows the percentage of time that specified discharges are equaled or exceeded.

Flow, Free Surface. Flow of water in which an interface exists between air and water.

Flow, Gradually Varied. Varied flow in which the velocity or depth changes gradually over a long distance.

Flow, Laminar. Flow of a fluid in which the viscous forces are predominant. In channel flow the fluid particles move approximately in definite, relatively smooth paths with no significant transverse mixing. In channel flow it occurs at Reynolds number smaller than 500-2000 and in flow through porous media at Reynolds number smaller than 1-10.

Flow, Nonuniform. Flow in which the velocity vector is not constant along every streamline.

Flow, Open Channel. Flowing water having its surface exposed to the atmosphere.

Flow, Rapidly Varied. Varied flow in which the velocity or depth changes abruptly over a comparatively short distance.

Flow Regime. A range of flows producing similar bed forms, resistance to flow, and mode of sediment transport.

Flow, Sheet. Flow in a relatively thin sheet, of nearly uniform thickness over the soil surface.

Flow, Steady. Flow in which the velocity is constant in magnitude or direction with respect to time.

Flow, Turbulent. Flow with turbulence. In channel flow, it occurs at Reynolds number larger than approximately 5000.

Flow, Uniform. Flow in which the velocity vector is constant along every streamline.

Flow, Unsteady. Flow in which the velocity changes in magnitude or direction with respect to time.

Flow, Varied. Flow in which velocity or depth changes along the length of the channel.

Fluvial Sediment. Fragmentary material that originates from weathering of rocks and is transported by, suspended in, or deposited from water.

Freshet. Flooding or overflowing of a stream caused by heavy rains or snowmelt.

Gage Height. The water-surface elevation referred to some arbitrary gage datum. Gage height is often used interchangeably with the more general term stage although gage height is more appropriate when used with a reading on gage.

Gaging Station. A particular site on stream, canal, lake, or reservoir where systematic observations of gage height or discharge are obtained.

Geology. The science which treats of the earth, the rocks of which it is composed, and the changes which it has undergone or is undergoing.

Geomorphology. The study of the characteristics, origin, and development of land forms.

Habitat. The region where animals or plants naturally or usually live or are found.

Hydraulic Jump. Sudden passage of water in an open channel from supercritical depth to sub-critical depth, accompanied by energy dissipation.

Hydraulic Radius. The cross-sectional flow area of a conduit divided by its wetted perimeter.

Hydraulics. The science treating of the laws governing water or other liquids in motion and their applications in engineering.

Hydrograph. A graph showing stage, flow, velocity, or other properties of water with respect to time.

Hydrostatics. The statics of fluids, usually confined to the equilibrium and pressure of liquids.

Islands. The vegetated areas within the channel banks separated from the mainland by the main channel and side channel.

Levee. Water-retaining earthwork used to confine streamflow within a specified area along the stream or to prevent flooding due to waves or tides.

Levee, Natural. Low alluvial ridge adjoining the channel of a stream, composed of sediment deposited by flood water which has overflowed the banks of the channel.

Load (Sediment Load). The sediment that is being moved by a stream. (Load refers to the material itself and not to the quantity being moved.)

Load, Bed. That part of the total sediment load that moves by rolling or sliding along the bed.

Load, Bed-Material. That part of the total sediment load which is composed of grain sizes represented in the bed--equal to the transport capacity of the flow.

Load, Suspended. That part of the total sediment load that is supported by the upward components of turbulence and that stays in suspension for an appreciable length of time.

Load, Total Sediment. The sum of the bed-material load and the wash load, or bed load and suspended load, or measured and unmeasured load.

Load, Wash (Fine Material). That part of the total sediment load which is composed of particle sizes finer than those represented in the bed--determined by available bank and drainage area supply rate.

Lower Flow Regime. A category for flows producing bed forms of ripples, ripples on dunes, or dunes. In this flow regime, flow is tranquil, water-surface undulations are out of phase with bed undulations, and resistance to flow is large.

Meander. One curved portion of a sinuous or winding stream channel, consisting of two consecutive loops, one turning clockwise and the other counterclockwise.

Meander Belt. That part of the valley floor situated between two parallel lines tangential to successive, fully developed meanders at their extreme limits.

Meander Length. Distance along the river between two corresponding points at the extreme limits of two successive, fully developed meanders.

Meander Width. Amplitude of swing of a fully developed meander, measured from midstream to midstream.

Measured (Sampled) Zone. 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 0.3 ft of the bed. Above this point is termed the sampled or measured and below unmeasured zone.

Median Diameter. The midpoint in the size distribution of sediment such that half the weight of the material is composed of particles larger than the median diameter and half is composed of particles smaller than the median diameter.

Morphology, Fluvial. Science of formation of beds and flood plains and of forms of streams by the action of water.

Ox-Bow. Abondoned part of a former meander, left when the stream cut a new, shorter channel.

Pelagic Community. A group of plants or animals living within water columns.

Plane Bed. A bed form in which there are no irregularities larger in amplitude than a few grain diameters.

Point-Integrated Sample. A water-sediment mixture that is accumulated continuously in a sampler that is held at a relatively fixed point in a stream and that admits the mixture at a velocity about equal to the instantaneous stream velocity at the point.

Pool. A deep reach of a stream. The reach of a stream between two crossings. Natural streams often consist of a succession of pools and crossings.

Reach. 1. The length of a channel for which a single gage affords a satisfactory measure of the stage and discharge. 2. The length of a river between two gaging stations. 3. More generally, any length of river.

Recurrence Interval (Return Period). The average interval of time within which the given flood will be equaled or exceeded once.

Regime. "Regime theory" is a theory of the forming of channels in material carried by the streams. As used in this sense, the word "regime" applies only to streams that make at least part of their boundaries from their transported load and part of their transported load from their boundaries, carrying out the process at different places and times in any one stream in a balanced or alternating manner that prevents unlimited growth or removal of boundaries. A stream, river, or canal of this type is called a "regime stream, river, or canal." A regime channel is said to be "in regime" when it has achieved average equilibrium; that is, the average values of the quantities that constitute regime do not show a definite trend over a considerable period--generally of the order of a decade. In unspecialized use "regime" and "regimen" are synonyms.

Riffles. Shallow rapids in an open stream, where the water surface is broken into waves by obstructions totally or partly submerged.

Ripples. Small triangular-shaped bed forms that are similar to dunes but have much smaller and more uniform amplitudes and lengths. Wave lengths are less than about 2 ft, and heights are less than about 0.2 ft.

River Bed. Lowest part of a river valley shaped by the flow of water and along which most of the sediment and runoff moves in interflood periods.

River Mile. River mile of a section is the mileage between the section and a reference point along the river thalweg or main-flow path.

River Training. Engineering river works built in order to direct the flow, or to lead it into a prescribed channel, or to increase the water depth for navigation and other uses.

River Width. The distance between vegetated banks taken normal to the general direction of flow in the river.

Sand. Sediment particles that have diameters between 0.062 and 2.0 mm.

Sandbar. A dune-shaped bed form whose upstream surface is extremely long in relation to the geometry of the channel (length, 2-3 times the width of the channel). The bar may often protrude above the flow.

Sand Waves. Crests and troughs (such as ripples, dunes, sandbars, antidunes, or standing waves) on the bed of an alluvial channel that are formed by the movement of the bed material.

Scour. Erosive action--particularly, pronounced local erosion--of water in streams, in excavating and carrying away materials from the bed and banks.

Sediment. Fragmental material that originates from weathering of rock and is transported by, suspended in, or deposited by water or air.

Sediment Concentration. The ratio of dry weight of sediment to total weight of the water-sediment mixture, expressed in parts per million.

Sediment Discharge. The amount of sediment that is moved by water past a section in a unit of time.

Sediment Yield. The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified period of time. This is equal to the sediment discharge from the drainage area.

Shear Stress. The internal fluid stress which resists deformation.

Shingle. Gravel and cobblestones deposited by water to resemble lapped roofing pieces. The origin is "shingl"--a Norwegian term for a small round stone.

Sieve Diameter. The size of sieve opening through which the given particle will just pass.

Silt. Sediment particles whose diameters are between 0.004 and 0.062 mm.

Sinuosity. The ratio between thalweg length to down valley distance.

Stage. The height of a water surface above an established datum plane, also gage height.

Standing Waves. Curved symmetrically shaped waves on the water surface and on the channel bottom that are virtually stationary. When standing waves form, the water and bed surfaces are roughly parallel and inphase.

Stream. A general term for a body of flowing water. In hydrology the term is generally applied to the water flowing in a natural channel as distinct from a canal. More generally as in the term stream gaging, it is applied to the water flowing in any channel, natural or artificial. Streams in natural channels may be classified as follows:

Perennial. One which flows continuously.

Intermittent or Seasonal. One which flows only at certain times of the year when it receives water from springs or from some surface source such as melting snow in mountainous areas.

Ephemeral. One that flows only in direct response to precipitation, and whose channel is at all times above the water table.

Stream Discharge (Water Discharge). The quantity of natural water passing through a cross section of a stream in a unit of time. (The natural water contains both dissolved solids and sediment.)

Streamline. Line envelope in space of the tangents to the instantaneous flow direction at a given time.

Stream Order. Number expressing the degree of branching in a stream system.

Stream Tube. Surface formed by streamlines passing through a closed curve (which is not a streamline).

Surface Areas, River. The area between the vegetated riverbanks.

Surface Areas, Riverbed. The river surface area less the area of the islands.

Tailwater. Water located just downstream from a hydraulic structure on a stream.

Terrace. A berm or discontinuous segments of a berm, in a valley at some height above the flood plain, representing a former abandoned flood plain of the stream.

Thalweg. Line following the deepest part of a streambed or channel or of a valley.

Transition. A category for flows that occur between the lower and upper flow regimes and produce bed forms ranging from those typical of the lower flow regime to those typical of the upper flow regime.

Trap Efficiency. Ability of a reservoir to trap and retain sediment. Expressed as a percent of sediment yield (incoming sediment) which is retained in the reservoir.

Trend. Unidirectional, monotonous (diminishing or increasing) change in the average value of a hydrological variable.

Turbidity. Condition of a liquid due to fine, visible material in suspension which impedes the passage of light through the liquid.