Inland Navigation Channel Training Works
ASCE MOP

Task Committee Members

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CONTENTS

PREFACE

This manual was produced by a Task Committee of the Waterways Committee of the Coasts, Oceans, Ports, and Rivers Institute (COPRI), American Society of Civil Engineers. Members of the Task Committee are:

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Eleven of the thirteen chapters were assembled by Bruce McCartney and Tom Pokrefke with input and critical reviews from the other Committee members, several U.S. Army Corps of Engineers offices, and existing publications. Bruce McCartney is the principle author of Chapter 1, William McAnally is the principle author of Chapter 2, Bruce McCartney, Tom Pokrefke, and Freddie Pinkard are the principle authors to Chapter 4, Freddie Pinkard and Dave Gordon added significantly to Chapter 5, and Freddie Pinkard and Steve Ellis are the principle authors of Chapter 6. Dave Gordon added significantly to Chapter 7, Mike Cox and Freddie Pinkard contributed significantly to Chapter 8, and Tom Pokrefke is the principle author of Chapter 11 with the assistance of Dave Gordon. Tom Pokrefke is the principle author of Appendix A with input from the other Committee members. Steve Ellis located the information in Appendix B for the manual. The authors extend their appreciation to Carol A. McAnally for her editorial reviews.

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CHAPTER 1

INTRODUCTION

1.1 PURPOSE

This manual presents design guidance on various types of waterway training structures. These structures confine and stabilize a channel by reshaping it to create reliable depths and widths for safe and dependable vessel transit. The goal of river training systems is to provide the desired channel dimension and alignment with little or no maintenance dredging. The proper use of dikes and revetments can achieve this goal as shown by Figure 1.1. Navigation channels can be in rivers (one flow direction) or in coastal areas (reversing tidal flow). This manual will be confined to river systems. These river channels are subdivided into “open river” and lock and dam systems built in existing rivers. Information on tidal channel training structures can be found in ASCE Manual of Practice No. 107, Ship Channel Design and Operation (see McCartney et al, 2005). Appendix A provides a terminology list of numerous terms related to channel training works presented in this manual.

FIGURE 1.1 Photograph showing the use of dikes and revetments on the Mississippi River

1.2 NAVIGATION ENGINEERING

This manual joins a larger body of literature in the field of “Navigation Engineering” (NE). This newly, evolving civil engineering discipline is defined as:

The practice of life-cycle planning, design, construction, operation, and maintenance of safe, secure, reliable, efficient, and environmentally sustainable navigable waterways (channels, structures, and support systems) used to move people and goods by waterborne vessels.

This manual joins four other ASCE manuals on NE subjects. The other NE manuals in the ASCE Manuals and Reports on Engineering Practice are:

No. 50, Planning and Design Guidelines for Small Craft Harbors (see ASCE, 1994}

No. 94, Inland Navigation: Locks, Dams and Channels (see McCartney et al, 1998)

No. 107, Ship Channel Design and Operation (see McCartney et al, 2005)

No 116, Navigation Engineering Practice and Ethical Standards (see McAnally, 2009)

These manuals can be accessed using the following link:

http://www.asce.org/BookstoreSeries.aspx?id=2147486137&series=Manuals%20of%20Practice%20(MOP)

The U.S. Army Corps of Engineers (USACE), Engineer Manual series and PIANC publications also provide information on NE subjects. The Corps Engineer Manuals may be accessed using the link below:

http://140.194.76.129/publications/

while the PIANC publications may be access using the link below:

http://www.pianc.org/publications.php

1.3 SCOPE

This manual will focus on the "Training Structures" component of navigation channel design in mainly open river channels; however, many of the types of structures presented can and have been used in lock and dam systems. The information in this manual is applicable to "Open River" projects (no locks and dams) and "Run of the River" projects (low-head locks and dams with no reservoir storage capacity). High-head locks and dam projects have deep upstream pools and do not need channel training structures except for the reach immediately downstream from the lock structure. Channel width, depth, alignment, and maintenance dredging are covered in ASCE Manual of Practice No. 94. Subjects included in this manual are: planning, design, construction, cost, repair, environmental impact, model studies, and case histories.

1.4 TRAINING WORKS ON MAJOR RIVER SYSTEMS

To gain an understanding of the vast application of river training structures, it is enlightening to see the magnitude of riverine systems within the United States that have such structures (Figure 1.2). The ASCE 2009 Report Card for American Infrastructure, Inland Waterways (see ASCE, 2009) provided a good description of the federally operated and maintained Inland Waterway System as follows:

Mississippi River and Tributaries System	9,000 miles
Ohio River and Tributaries System	2,800 miles
Gulf Coast Intercoastal Waterways System	1,109 miles
Columbia River and Tributaries System	596 miles

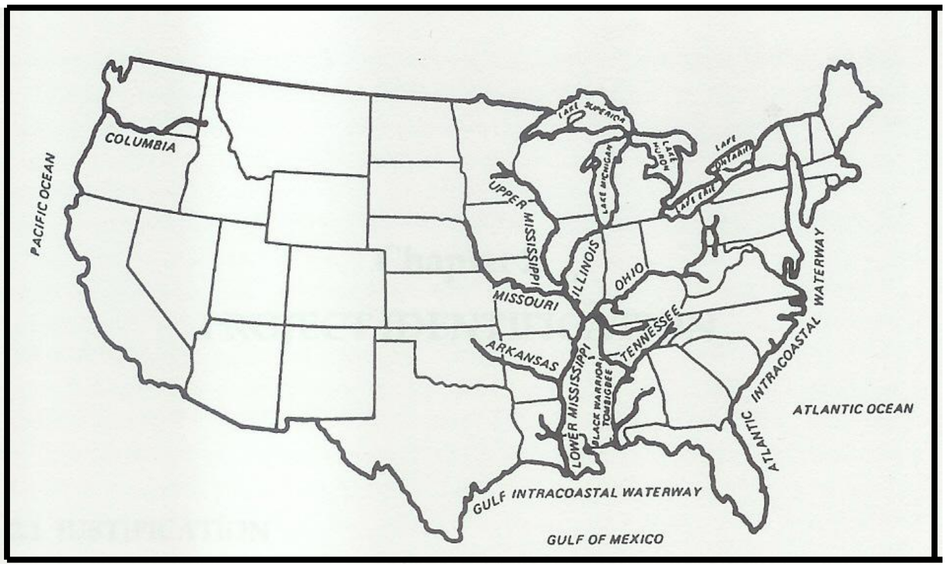

FIGURE 1.2 – U.S. Inland Waterway System

Since the Gulf Coast Intercoastal Waterway System is located in tidal salt waters with small tidal fluctuation, these training structures do not fit into the scope of this manual. The training structures on the remaining systems, Mississippi, Ohio, and Columbia are the focus of this manual; however, as will be seen in this manual, applications on other systems such as the Tennessee-Tombigbee, Red River in Louisiana, and other systems provide additional design information. The extent of training structure used depends on the type of navigation channel within a system. For example:

> The Columbia-Snake system has about 150 miles of open river from Bonneville Dam to the Pacific Ocean and about 450 miles of deep channels between eight (8) high lift locks having total lifts (from the downstream to upstream pools) ranging from 69 feet to 110 feet. Training works on the open river section are widely spaced timber pile dikes and sand and rock revetments. The channels between locks have stone revetments immediately downstream from the locks and dams for bank stabilization against high velocities. Once dam discharge reaches the wide, deep reservoir, the velocity slows considerably.

> The Mississippi River system, including tributaries, has a variety of channel types. The upper Mississippi River is controlled by run of the river low lift locks and dams with lock

lifts of 39 feet or less typically 10 to11 feet, excluding Upper St. Anthony Falls Lock and consisting of pools having no flood water storage capacity. This type of channel requires training/bank protection works for most of its length. Stone dike fields and stone revetments are the predominate type of control works used.

The Illinois Waterway (IWW), part of the Mississippi system, is primarily controlled by run of the river low lift locks and dams; however, there is one spillway type dam along the upper IWW near Chicago. Lock lifts vary from 4 to 39 feet and consist of pools that have no flood water storage capacity. This river provides much slower flows than the Mississippi River; therefore training works are not as effective as control structures. There are few training works along the IWW; revetments are the predominant control structure and are used for bankline and levee erosion reduction.

The Mississippi River from St. Louis, MO to the Gulf of Mexico is open river (no locks). Training works are normally articulated concrete mats (ACM) revetments and dike fields. Some river reaches employ stone revetments.

The Missouri River navigation channel (part of the Mississippi system) is open river (no locks) for its entire length. Stone dike fields and revetment are the primary means of channel training.

The Ohio River system is controlled by low lift locks and dams. This run of the river system is similar to the upper Mississippi River with stone dike fields and revetments as the predominant control structures.

1.5 INLAND WATERWAYS VALUE TO THE NATION

The Inland Waterway System is vital to the nation for economic well being and military logistics. The ASCE 2009 Infrastructure Report Card presents the economic importance:

"*Inland and intercoastal waterways directly serve 38 states as well as states on the Atlantic seaboard, the Gulf Coast, and the Pacific Northwest. Shippers and consumers in these states depend on the inland waterways to move approximately 630 million tons of cargo valued at more than $73 billion annually*."

The waterways strategic value to the military is the ability to move material and resources to inland bases or port deployment locations.

Although river training works consisting of dikes and revetment are relatively "unseen" attributes on the various river systems across the United States, these structures do impact the vast majority of the population over all of the major waterways, consist of a large number of individual structures and types, and are a significant investment for the Federal government. Therefore, it is imperative that these structures be planned, designed, constructed, and maintained in a safe, cost effective and environmentally sustainable way to continue providing the necessary training on waterways across the U.S. The purpose of this manual is to provide the river training structure designers with the best available design guidance and information to accomplish that task.

The 2009 ASCE Report Card also reported that the average age of the 257 navigation locks is nearly 60 years and 47% of all locks maintained by the U.S. Army Corps of Engineers were classified as being functionally obsolete in 2006. It is estimated that it would require more than $125-billion to replace all of the 257 locks. Assuming training works structures are 5% to 10% of system cost, the training works replacement cost would be at least $6-billion to $12-billion.This sizable investment warrants prudent design and repair considerations.

1.6 ORGANIZATION OF THIS MANUAL

This manual is organized into 13 Chapters and two Appendixes to show the elements needed to develop and maintain a functional navigation channel training works system. Individual chapters cover the following:

Chapter 1 – Extent of inland navigation system, value to the nation, and define the new Engineering specialty "Navigation Engineering"

Chapter 2 – Overview of sedimentation problems that need to be managed for a viable navigation project.

Chapter 3 – History of training works evolution in the U.S.

Chapters 4 through 7 – Training structures, types, layout, and design guidance.

Chapter 8 – Case studies of five U.S. waterways

Chapter 9 – Cost for initial construction and repairs.

Chapter 10 – Environmental design to improve aquatic habitat.

Chapter 11 – Model studies for training works layout or modification.

Chapter 12 – Various inspection methods to evaluate performance and identify repair needs.

Chapter 13 – Examples of repair techniques.

Appendix A – Terminology.

Appendix B – An analytical method to determine dike length.

CHAPTER 2

SEDIMENTATION AND SEDIMENT MANAGEMENT IN RIVER CHANNELS

2.1 GENERAL

In addressing the design of inland navigation channel training works it is appropriate to discuss riverine sediments and sediment management. This chapter provides the river engineer with a cursory view of the topic of riverine sedimentation and some of the dynamics associated with the movement of sediments in a river channel. Numerous books and text books are widely available that cover the subject of sedimentation in great detail and provide the engineer with in-depth information on sedimentation. Since the main purpose of training works is to impact and influence the movement, erosion, and deposition of the sediments passing in and around the navigation channel, a relatively brief summary is presented herein to help the river engineer appreciate the variations and complexity involved when dealing with river sediments. Sections of this chapter are adapted from McCartney et al. (1998) and USACE (1995).

Sediment, consisting of rock, mineral, and shell fragments plus organic materials, is naturally present in streams, rivers, lakes, estuaries, and ocean waters. It makes up the bed and banks of those water bodies, and flowing water transports it from place to place until it deposits. Some waters contain small amounts of sediment that are nearly invisible, while others contain so much sediment that the water becomes a chocolate brown. Visibility of the sediment also depends on how the water transports it. The nature and amount of the sediment and the flow determine whether the sediment is transported along the bed or suspended higher in the water column.

Waterborne sediment is a valuable resource. Deposited on a river's floodplain, it forms rich farmland such as the Mississippi Delta between Memphis and Vicksburg. Sand and gravel deposits in rivers and ancient river courses provide construction materials. Some aquatic species, ranging from tiny daphnia to sturgeon, thrive in high levels of suspended sediment, and gravel bed material serves as critical habitat for fish spawning. Along coastlines, sediment deposits build land and marshes that protect against flooding and offer productive habitat for aquatic species.

Despite its resource value, too much sediment or the wrong kind of sediment can cause economic and environmental damage. For example, deposits of fine-grained sediments on gravel bars can kill mussels and fish eggs (or cause them to move to shallower areas), and flood borne sediment can bury farms and damage homes. Few port or waterway operators see too little sediment as a problem. Excessive sediment deposition in ports and channels reduces their depth, forcing vessel operators either to time transits to high water periods, to light-load so as to reduce draft, or to limit passage into unsafe narrow passages, or preventing access altogether. The traditional solution to these problems was dredging and disposal of excess sediment. More recently, the beneficial use of dredged sediment has been recognized for the value of this resource by using it for shoreline restoration, marsh creation, and construction material, but usually at increased cost to those performing the dredging (PIANC 1992). Disposal, other than beneficial uses, has become constrained, with in-water placement often prohibited and on-land placement options diminishing.

Waterborne sediment can be classified by size of the primary grains, from largest to smallest, into boulders, cobbles, gravel, sand, silt, and clay. Larger sizes move mainly by rolling, sliding, or hopping along the bed only when the water is moving swiftly; whereas, finer sizes and organic materials move in suspension throughout the water column. Sizes in the middle may move in either or both modes, depending on the water flow and bottom configuration. Sand-sized (grain diameter greater than 0.062 mm) and larger particles are noncohesive, so they move nearly independently of other particles. Because they are relatively large, they may settle very rapidly to the bed when flow slows down or stops. Clay particles are tiny (grain size 0.004 mm and smaller), and they tend to stick together (flocculate) and move as aggregates of many individual grains. They may settle very slowly, even in quiet water. Silt, falling between sand and clay in size, may behave either like sand or like clay. Organic materials include plant and animal detritus and settle very slowly and may help bind sediment grains together.

Cohesion of sediment particles influences bed behavior also. New clay deposits are usually porous and easily resuspended. With time and overburden pressure, clay deposits consolidate and become denser and more resistant to erosion.

2.2 SEDIMENT TRANSPORT

Sediment is transported from one place to another by flowing water. Depending on the size and degree of cohesion of the sediment grains and intensity of the flow, the amount transported may be proportional to the velocity of the flow or proportional to the velocity squared, cubed, etc. So a doubling of flow velocity may increase sediment transport as much as eight-fold. In some cases more sediment is transported in one storm event than in all the rest of the year.

The proportionality effect described above can also cause substantial sediment deposition. If a waterway's cross-section is suddenly increased by increased depth or width such as when the stage goes above bankfull, the flow velocity drops and the capacity to transport sediment falls even faster, so sediment will tend to deposit. This effect is a common cause of shoaling in navigation channels and ports, and is sometimes used to force sediment deposition in a particular location, such as sediment trap.

Vessel traffic can suspend sediment from the bed and banks of a waterway through:

- Flow under and around the vessel as water moves from the bow of the vessel to the stern
- Pressure fluctuations beneath the vessel
- Propwash striking the bed
- Bow and stern waves agitating the bed and breaking against the bank

Figure 2.1 illustrates the surface sediment plume that can form due to vessel passage. Sediment suspended by vessel traffic can either quickly settle out (if the sediment consists of sand-sized material) or remain in suspension (if the sediment consists of very fine silts or clay-sized material). A fine sediment suspension has greater density than the surrounding water, so it can flow as a density current away from the point of suspension. The latter process can move sediment from the waterway centerline into relatively quiet berthing areas, where it settles out. This phenomenon has been documented in several locations (e.g., Kelderman, et al. 1998).

FIGURE 2.1 Surface sediment plume from vessel passage

Eddies, circular flow patterns formed by flow past an obstruction or in front of an opening like a port basin, have a complex three-dimensional circular structure with flow inward near the bottom and outward near the surface with a quieter zone in the middle. Sediment passing near an eddy is drawn into the eddy and pushed toward the center, much like loose tea leaves in a stirred cup, where it tends to deposit. This phenomenon is a common cause of sedimentation in slips, side channels and berthing areas.

Natural streams can be characterized by their tendency to meander and migrate, irregularities and changes in geometry, varying stage and discharge, and variations in the composition of beds and banks. Many of the problems encountered in the development and improvement of natural streams are concerned with channel alignment and the movement of sediment into and within the stream. Scouring of the bed and banks and deposition in critical areas can affect channel depth, width, and alignment and the operation and use of facilities and structures for navigation such as locks, harbors, docking areas, and other facilities such as hydropower plants, sewage systems, and water intakes. Sediment movement can also affect the capacity of the channel to pass flood flows.

2.3 SEDIMENTATION PROBLEMS

River sediment problems associated with navigation can be grouped into two main categories; local scour and deposition, and general degradation (overall erosion) or aggradation (deposition) problems. The primary local sediment problems are deposition in navigation channels and pools, degradation below dams, and streambank erosion. The second are the stream's response to changes in the discharge hydrograph and sediment supply by a number of possible causes, including upstream land use and navigation projects. Vanoni (1984) and Garcia (2008) provide an extensive discussion of stream morphology and sedimentation processes.

Alluvial rivers tend toward a dynamic equilibrium condition balancing the water and sediment loads imposed upon them. Any significant modifications to the system (realignments, locks, dams, etc.) will disrupt this balance and a period of adjustment will occur as the stream attempts to reestablish a new state of dynamic equilibrium. During this period of adjustment, which in some cases can last years, decades, or longer, sediment-related problems may be increased. It is important to recognize that rivers in so-called equilibrium do not have static geometry or sediment properties. Due to normal variations in the annual hydrographs and sediment inflow conditions, specific locations on a river in dynamic equilibrium may deposit at high flow and scour at low flow while the opposite may be occurring at another location, e.g. bends and crossings.

Development of a river system for navigation may involve the construction of several major components such as locks and dams, bank stabilization, river training structures, reservoirs, and channel realignments. The impacts of each of these components of work can be assessed individually. However, the ultimate channel response depends on how the system integrates these individual impacts in an effort to attain a new equilibrium state. Because of this complexity it is difficult to develop definite rules or trends that apply to all navigation projects or all rivers. Design criteria and techniques that have been successful on one river system may not be feasible on another system which has different hydrologic, sediment, or geomorphic characteristics.

Common problems associated with high-head dams, with a head difference of 30 feet or more, are aggradation in the upper pool followed by degradation of the downstream channel. Low-head dams generally follow somewhat different trends, since they are designed to allow open-river conditions during the high-flow periods when the majority of sediment is transported. Special care must be taken to ensure that open-river flows occur frequently enough so that the existing sediment transport regime is not unnecessarily altered.

Natural shoaling problems affecting channel width, depth, and alignment can be encountered in any stream carrying sediment. These problems can usually be expected in crossings, long straight reaches, or bendways where the low-water channel tends to be unstable, at mouths of tributary streams, in reaches where there is divided flow or bifurcated channels, in lock approaches, and in entrances to slack-water canals or harbors. Most shoaling problems are local and solution of these problems requires knowledge of the characteristics of the reach under study, the reach just upstream, to a lesser extent the reach just downstream, and the factors affecting the movement of sediment in these reaches. The design engineer should be concerned more with the sediment contributing to the problem, flows during which the problem or problems develop, and the principles involved in its development than in the total sediment load moving through the reach. Generally, the sediment forming the shoal is only a very small part of the total sediment load but can be sufficient to create problems for navigation (McCartney et.al., 1998).

Natural channels having erodible bed and banks will tend to meander, developing a sinuous course consisting of a series of alternate bends and crossings with some relatively straight reaches. The degree of sinuosity assumed by these channels depends on many factors including discharge, sediment load, valley slope, and composition of bed and banks. Unless the meandering of these channels is resisted by bank stabilization and training works, the bends will tend to migrate and change through the erosion and caving of their banks and the process of channel erosion and deposition. The major concern on actively meandering streams is that this process can be very dynamic and major changes in the channel alignment can occur rapidly and dramatically. The meandering process often creates channel alignments which have poorly aligned navigation channels that are unstable and may vary significantly over short time periods. The channel depth in both active and stabilized meandering channels is deeper in bends along concave banks and shallower in crossings and straight reaches. Refer to Figure 4.11 for an illustration of river channel alignment morphology. During rising stages (high energy) bendways scour and crossings fill. During falling stages (low energy) crossings scour and bendways fill. Also, as the discharge increases flow tends to move straighter down valley which scours additional material off of the point bar (inside of the bendway) adding even more sediment to be deposited in the downstream crossing. The increase in depth can be as much as one half to more than the amount of increase in stage, depending on the curvature of the bend and alignment of the channel upstream. With other conditions remaining the same, the increase in depth appears to be more a function of the river stage and stage duration, than of the rate of change in stage (McCartney et.al., 1998).

The scouring of the channel in bends may cause a large amount of sediment to move into the crossing and reach just downstream. Because of the concentration of high-velocity currents and turbulence in bends, much more sediment can be moved in time in sinuous channels than may be moved in straight channels with the same average velocity and slope. For this reason straight channels and crossings downstream of a bend will tend to be shallow and unstable. Low-water slopes through bends are generally lower than the average because of the backwater effect produced by the shallow crossing downstream. Because of the reduced slopes and velocity, deposition occurs in bends while scour occurs in crossings during the low flows. However, the amount of deposition in the bends is seldom sufficient to reduce depths to less than that required for navigation (McCartney et.al., 1998).

In meandering waterways the low-water channel in the straight reach between alternate bends crosses from one side of the river to the opposite side. Because the movement of sediment in a bend is greater than the capacity of the straight channel downstream during the higher-flows, deposition occurs in the crossing, limiting depths and at times widths available for navigation. As river stages decrease, slopes and velocities over the crossing tend to increase, increasing the movement of sediment and depths. The rate of scour and depths available for navigation depend on the stage, stage duration, and rate of recession from high to low stages. After a prolonged high-water period or after a rapid decrease in stage, depths in crossings will tend to limit navigation depths available and are a frequent source of navigation difficulties. Alignment and depth of the channel in crossings depend on variations in flow conditions and alignments of the reaches upstream and downstream. Maintaining a satisfactory channel in crossings will be more troublesome if regulating structures on the concave side of the bend upstream are not carried far enough downstream to prevent dispersion of the higher flows and if the crossings to the next bend are relatively long. Extending the training works in a bend toward the crossing downstream improves the alignment and depth of the channel over the crossing and flow into the bend downstream. The complexity in using such river training structures is compounded due to the various types of structures available and the various impacts the structures can have on sediment movement and deposition (McCartney et.al., 1998).

Channels in long straight reaches or in long flat bends will tend to meander within their banks. Development and maintenance of a satisfactory channel through these reaches are more difficult than in a sinuous reach and could be affected by variations in discharge, relative sediment-carrying capacity of the reach upstream, or sand waves moving through the reach. Unstable reaches will tend to have a higher low-water slope than will stable reaches.

Bifurcated channels or divided flow will be found in many alluvial streams in addition to those formed by cutoffs, and when flow is divided into separate channels the overall sediment transport capacity is typically reduced. Side channels may tend to carry a greater proportion of the sediment load than the proportional discharge, because of the lateral differential in water level which depends on the shape, size, and angle of entrance with respect to the direction of flow from upstream and the relative lengths of the two channels. When the entrance to the side channel is wide in comparison with the rest of the channel, sediment will tend to be deposited near the entrance, which could eventually reduce or eliminate flow through the channel during low stages. Depths in the main channel will tend to be limited when side channels carry a sizable proportion of the total flow, and the partial or full closure of these channels will be required to improve depths in the main channel. When deposition occurs near the entrance, the sediment-free flow moving downstream of the entrance could cause scouring and deepening of the side channel and bank caving. When there is a substantial amount of flow diverted through a side channel, the main low-water channel will tend to develop toward the point of diversion (McCartney el.al., 1998).

Flow from tributary streams causes a local increase in water level just upstream and channelward of the inflow and a lowering of the water level along the adjacent bank downstream. The difference in water level will depend on the discharge, and current direction and velocities of the tributary flow entering the main stream. Because of the lateral differential in water level created, there will be a tendency for shoaling along the adjacent bank downstream and for sediment carried by the tributary to be moved along that side of the channel. Accordingly, the deeper channel will tend to form away from the adjacent bank (McCartney et.al., 1998).

Entrances to canals and slackwater harbors involve openings in the bank line and a local increase in the channel width. This causes a lowering of the water level at the entrance and a tendency for bottom currents and sediment to move toward the entrance, resulting in a tendency for shoaling. The amount of shoaling will depend on the amount of sediment carried by the stream, size variations of the sediment, size of the entrance, and location of the entrance with respect to the alignment of the stream channel. Shoaling in the entrance could also be affected by the rate of rise and fall of river stages that cause flow toward and away from the canal or harbor.

2.4 SEDIMENT MANAGEMENT

Sediment management employs engineering solutions and natural transport processes to minimize adverse economic and environmental effects of too much or too little sediment. It is typically accomplished at the local scale – from meters to a few kilometers of a single project – but should be viewed in a regional context. Regional sediment management recognizes that locally optimum solutions may have adverse effects downstream or even upstream from the site. For example, bulkheads along an eroding bankline may prevent local erosion, but increase erosion downstream if the river's capacity to transport is not balanced with the sediment supply.

A variety of engineered solution approaches to reduce deposition problems are available. Solutions tend to be unique to each channel, for a successful design depends on waterway configuration, flow conditions, and sediment type and supply; however, all solutions can be placed in three categories — Prevention, Treatment, and Accommodation, as shown in Table 2-1.

Prevention can be accomplished by one of three strategies listed in the Table – Keeping Sediment in Place (KSP), Keeping Sediment Out (KSO) and Keeping Sediment Moving (KSM). KSP is the most desirable, for it aims to prevent land and bank erosion at its source, reducing the costs of both erosion and deposition, but may be impractical in some cases. KSO is more localized, aimed only at the problem area, and may not reduce the economic cost. For example, a sediment trap does not eliminate dredging, it simply locates the deposition in a place where dredging is more convenient, which may or may not be less expensive. Trap design is addressed below.

KSM keeps sediment in the channel moving (and prevents net deposition) and can be thought of as manipulating natural processes to manage deposition; however, it may increase deposition problems downstream. If suspended sediment can be kept suspended while the flow passes through the channel reach, or if the flow maintains high enough tractive force (usually expressed as shear stress, or drag force per unit area) to keep coarser particles moving as bedload, sediment will pass on through without depositing. Methods to keep sediment moving include:

- Structural elements that train natural flows
- Devices that increase tractive forces on the bed (such as water jet arrays)
- Designs and equipment that increase sediment mobility (such as water injection dredges)
- Designs that reduce cohesive sediment flocculation (useful only in very quiet pools behind dams)

The Treatment category includes Keeping Sediment Navigable (KSN), which employs the nautical depth concept used mostly in coastal waters (PIANC, 2008) and dredging. Dredging includes:

- Dredging and Dredge and Remove Sediment (DRS)
- Dredging and Dredge and Place Sediment (DPS)
- Agitation of deposits so that the sediment is transported downstream

Table 2-1. A Sediment Deposition Management Taxonomy for Channels

CATEGORY	**STRATEGY**	**EXAMPLES**
Prevention	KSP – Keep Sediment in Place	Erosion control on land and/or bed and banks
	KSO – Keep Sediment Out	Sediment Traps, Gates and Dikes, Channel Separations
	KSM – Keep Sediment Moving	Training Structures, Agitation, Flocculation Reduction , Flushing

		Flows
Treatment	KSN – Keep Sediment Navigable	Nautical Depth Definition, Aerobic Agitation
	DRS – Dredge and Remove Sediment	Placement in confined disposal facilities or offshore, Permanent beneficial uses
	DPS – Dredge and Place Sediment	Bypass sediment (KSM), Temporary beneficial uses
Accommodation	Adapt to Changing Sediment Regime	Flexible infrastructure, light loading, seasonal navigation

2.5 SEDIMENTATION STUDIES

Potential sediment problems may be minimized, and in some cases prevented, by conducting a detailed sedimentation study of the stream system before designing a project. As one component of a comprehensive geomorphic analysis, the sedimentation study is aimed at developing an improved understanding of the significant sedimentation processes within the basin. The major emphasis of this type of study should be on analyzing the channel morphology and sedimentation phenomenon during the historic period, although longer term system changes are also considered. As a minimum the sedimentation study should document the variations in sediment transport (size and quantity), identify all major sources of sediments (bed and banks, tributaries, etc.), locate degrading, aggrading, and stable reaches, and establish the range of flows transporting the majority of sediments. Correlating the results of the sedimentation study with historical changes in the basin (channel improvements, land use, reservoirs, etc.) enables the engineer to develop a firm understanding of past and present sedimentation processes. With this information the effects of anticipated project features can be analyzed qualitatively. The river engineering designer should realize that the prototype provides a wealth of information and in fact provides the designer with an initial glance of what may or may not work as an improvement plan is developed. Determining reaches that are stable in depth and width provides the designer with initial or target parameters to initiate the design. Likewise in bendways, determining radius of bends that provide adequate navigable width provides additional design data. From there the river engineering designer can undertake various types of model studies to verify and/or enhance the design parameters developed initially. A qualitative analysis of this nature is essential for the development and interpretation of results from sediment transport models.

2.6 SEDIMENT DATA NEEDS

Knowledge of sediment characteristics and transport is essential for the design of river engineering works on alluvial streams. Basic data collection should include suspended sediment samples, bed-load samples (if possible), bed and bank material samples, and borings in the streambed and banks. Sampling stations should not be restricted to the limits of the navigation project but should include upstream and downstream reaches, as well as major tributaries. Analyses of samples should include complete grain size distributions and standard geotechnical tests, such as liquid limit. Fine-grained sediment requires additional testing, as described by EM 1110-2-4000 (USACE, 1995).

CHAPTER 3

HISTORY

River training works construction in the U.S. began with the advent of river modification for waterborne commerce. The U.S. Army Corps of Engineers (the Corps) was charged with development of the inland waterway system as early as 1820. Examples of early channel improvements and dike construction are:

Missouri River -- 1832, snag removal

Upper Mississippi River -- 1870's, construction of wing dams (dikes) began

Lower Columbia River -- 1885, construction of timber pile dikes began

The Corps has continued this inland waterway development and maintenance from the 1830's to today. Most of the knowledge and experience for the U.S. inland waterways and river training works is recorded in Corps design manuals and other Corps publications.

During the colonial era, hydrographic and topographic surveys of the Ohio and Mississippi Rivers conducted by British and French engineers added immeasurably to the store of knowledge about the course and character of these great waterways. By the time of the American Revolution, New Orleans was developing into a market for the agricultural produce of the Ohio Basin. The 1803 Louisiana Purchase that gave the United States possession of the balance of French holdings in North America, secured this important port for the infant nation and advanced trade with eastern and foreign markets.

Commercial navigation was initially conducted with flatboats and keelboats. But the introduction of steam navigation to the Mississippi River in 1811 touched off an explosion of development. This technological device transformed a raw frontier of scattered habitation into a settled region of cultivated landscapes and burgeoning cities. These light-draft vessels, however, often fell victim to rocks, snags, shifting sandbars, as well as other hazards.

In 1820, Congress appropriated the sum of $5,000 for the preparation of a survey of the Ohio and Mississippi Rivers in anticipation of improving these rivers for navigation. In April 1824, President James Monroe signed the General Survey Act, authorizing the Army engineers to conduct surveys and planning studies for transportation projects that might enhance national defense and commerce. The following month, he signed a River and Harbor Act that appropriated $75,000 for improvement of the Ohio and Mississippi Rivers, including the removal of snags that endangered river traffic.

Until the Civil War (1861-1865), contrasting views on the constitutionality of Federal waterways, sectionalism, and political enmities often thwarted attempts by Army engineers to make the Mississippi and other rivers safe commercial arteries. Funding was sporadic as political imbroglios undermined systematic river improvements.

Nevertheless, visionary engineers advanced theories and proposals for ambitious riverine programs, spawning engineering controversies that continued into the twentieth century. In 1850, the Corps was directed by Congress to make surveys and reports on the Mississippi and Ohio Rivers with a view to preparing plans for navigation and flood control improvements. This report gained attention because of its advocacy of flood control reservoirs. That same year the

Corps began work on a study of the lower Mississippi River's hydrology and topography that was published in 1861. Their report, often called the Delta Survey, discussed river hydraulics and the effects of cut-offs, overflow basins, tributaries, outlets, levees, and crevasses. Its impact on subsequent river engineering in the United States can hardly be exaggerated, for the authors stressed the improvement of navigation and flood control by building levees.

For the time being, however, implementing new river improvement strategies was forestalled by the Civil War. Following the conflict, new surveys were conducted and gauges were placed in the Mississippi River to systematically monitor stages. Appropriations for improvements especially on the lower Mississippi River, nevertheless, lagged due to the slow process of reinstating the former Confederacy to the Union. But as the nation healed its wounds, Congress became more receptive to proposals for improving the nation's greatest waterway. With the close of the Civil War, national attention once again focused on surveying and improving the Mississippi River.

The American engineers had a rich fund of European experience to draw upon. As early as the seventeenth century, the Germans were protecting the banks of rivers with masses of brush formed into fascines (bundles). This method of bank protection, called *blesswerk,* was also used for bank and shore protection in Holland. By the eighteenth century, the Germans had virtually abandoned this type of bank protection and were focusing their efforts on developing various types of longitudinal and spur-dikes. On the lower Rhine River, considerable effort was also devoted to realigning the river from making cutoffs and diverting the stream from important localities. In France, complex systems of dikes were built on the convex and concave bends of rivers, and the Italians pioneered the use of cellular structures of brush and poles on the Po River (1862) that were allowed to fill with sediment. Therefore, most of the technical literature available to the Americans stressed the proper placement of dikes and similar open river works that would contract the channel, protect the banks, and improve navigation. This general approach influenced the thinking of Corps engineers as they began the task of managing the Mississippi River.

In 1837, General Robert E. Lee did some channel improvement work in the St. Louis Harbor. Later work modestly began with a $25,000 appropriation in 1872 to improve the main stem between the mouth of the Missouri and Illinois Rivers. With these funds, the Corps closed a chute on the Mississippi River with a stone dike founded on a layer of willow brush. In 1873, the Corps' work was expanded by a $200,000 appropriation that extended surveys and construction to the reach between Cairo and St. Louis on the Mississippi River with special emphasis on improving navigation at two locations, Sawyer Bend and Horsetail Bar. In conjunction with plans for the dike work, proposals were developed for "brush rafts or mattresses" in 1873.

The origin of the Articulated Concrete Mat (ACM) is traced to subaqueous concrete bank protection introduced about 1890 in Italy. The revetment consisted of concrete or terra-cotta blocks strung on wires. Thereafter, it was used on the Po, Nile, and Dee Rivers with good results. The blocks were about 25.4 cm wide, 102.2 cm thick, and weighed 11 to 14 kg each. The galvanized or lead-covered wires connecting the blocks were attached at intervals of about 1.8 m to an anchor pile buried in the bank, and the revetment was laid by means of a special floating scaffold in elements 4.6 to 9.1 m wide. The upstream and downstream extremities of the revetment were protected with fascines. Since the wire-joined blocks conformed to irregular subaqueous slopes, this system deserves recognition as the first articulated concrete mattress.

The concrete mattress innovation which seemed to most directly influence work on the Mississippi River was developed in Japan. In 1908, B. Okazaki devised a type of reinforced concrete flexible mattress and placed it in a reach of the Yubari River. Okazaki, as chief engineer of the Ishikari River Improvement Works, Sapporo, Japan, eventually created an experimental concrete casting yard at Bannazuro where the blocks for the mattresses were made. The main body of the mattress consisted of 15.2 cm thick concrete formed in 15.2 cm by 0.6 m blocks. The reinforcement for each block consisted of No. 12 galvanized steel wire. After casting in special molds, the blocks were knitted together by passing wires through two holes previously made in the middle plane of each block. The mat was launched by a special floating device consisting of a scaffold and pontoons. ACM's were first tried on the Mississippi River as an experiment in 1915. The early ACM design has been refined over the years to the system used today (see Chapter 6). The above was adopted from Robinson and Ethridge (1985).

Another new development in revetment design is the trench-filled revetment. Trench-fill revetment was used for the first time on the Mississippi River in 1975. Trench-fill revetment consists of a rock-filled trench (ditch), constructed along the upper portion of a caving riverbank. As bank scour cuts into or under the trench, the rock is self-launched to provide underwater protection. It is particularly advantageous where the channel has not developed sufficiently to place ACM. In construction of the trench-fill revetment the upper bank is first rough graded to a 1 on 3 slope and the trench is then excavated to grade and stone fill is placed as soon as possible. Subsequently, the upper-bank grading is completed and the upper-bank paving stone placed (see Chapter 6).

Timber piles were the most common type of dike material in the early days of river training works in the U.S. In recent years, there has been a shift to stone for both new and repair work; however, timber pile are still in use and are included in the training works system for the recently completed J. Bennett Johnston Waterway (see Chapter 8), The lower Columbia River navigation channel is an example of timber pile dikes which are still in use. The lower Columbia River pile dikes are actually a hybrid of an enrockment (rip-rap stone) placed along the lower section of the dike structure, and timber piles extending vertically through the enrockment for given distance above the design water plane. Figures 8.2 and 8.3 illustrate these key components of pile dikes constructed within the lower Columbia River.

The Columbia River navigation system consists of:

- A deep-draft navigation channel from the Pacific Ocean to the ports of Portland, Oregon, and Vancouver Washington which is 106 miles upriver.
- A barge channel from Vancouver to Lewiston, Idaho (about 500 miles). The barge channel is open river from Vancouver to the Bonneville Lock and Dam, River Mile 138. The Bonneville to Lewiston channel is contained in a series reservoirs created by 8 high lift locks and dams.

The pre-1885 channel had controlling depths of 12 to 15 feet at low water on several bars. As ship size increased the channel was enlarged as follows:

Date Authorized	Channel Depth (ft)	Channel Width (ft)
1878	20	

1899	25	
1912	30	300
1930	35	500
1962	40	500
2009	43	600

A river training dike construction program accompanied this channel enlargement sequence. In the program 285 timber training dikes were placed in numerous river reaches from the Pacific Ocean upstream to Bonneville Dam. Timber pile dike construction started in 1885 and continued in several phases through the 1960's. Almost all dikes are at least 50 years old and in varying states of deterioration. Additional details of this timber pile dike system are presented in the Chapter 8.

During 2011, the Portland District conducted a structural and functional assessment of 233 USACE pile dikes within the lower Columbia River. The assessment considered pile dike function both in terms of navigation and maintenance/improvement of shallow water habitat within the lower Columbia River. The 2011 assessment will be used for each pile dike as a basis for recommending repair, modification, removal, or taking no-action and letting the dike deteriorate. The process the Portland District used for conducting this pile dike evaluation is presented in Chapter 12.

CHAPTER 4

TRAINING STRUCTURE TYPES AND LAYOUT

4.1 GENERAL

When working with navigation channel works it is obvious that there are a wide range of techniques and options available to the designer. It should be realized that the development and control of the river channel depends on the final desired objective of the particular project. Therefore, selection of the most effective method to use among the many and varied size and shapes of dikes and revetments for a particular river situation is highly dependent on the education, knowledge, and experience of the designer. All alternatives are not necessarily applicable in all situations and need to be adapted to site conditions and the size of the stream being investigated. Some of the techniques are more suitable in large inland waterways, such as the Mississippi River system, while these same techniques may or may not be suitable for the smaller inland waterways, such as the Red River in Louisiana.

There are two types of waterway training structures: re-directive and resistive. Re-directive, as the name implies, is the use of the river's energy and managing the energy in a way that benefits the system i.e., enhance the navigation channel. A resistive structure acts to maintain the system as status quo i.e., reducing bank erosion.

4.2 RE-DIRECTIVE STRUCTURES

Re-directive structures are usually a series of dikes placed along the inside of a river bend where sediment usually deposits. Dikes function continually at lower river stages; however, the effects of dikes will decrease or "wash out" when overtopped at higher river stages. The major functions of dikes concentrate the river's energy into a single channel to control the location and increase the depth of the navigation channel and impact the erosional and depositional characteristics of the river to reshape the dimensions of the navigation channel. Dikes have been known by a variety of names throughout the years, such as groins (or groynes), contracting dikes, transverse dikes, cross dikes, spur dikes, spur dams, cross dams, wing dams, and spurs. The most common dikes in use today are shown in Figure 4.1.

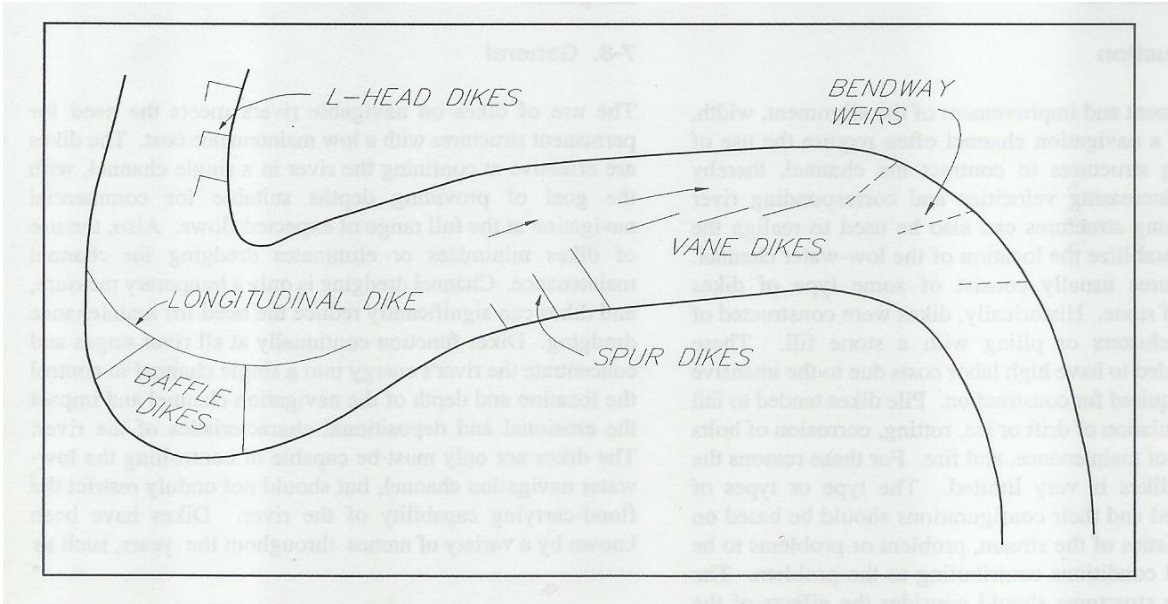

FIGURE 4.1 Types of dike structures

Certain types of control works are essential in the very early phases of development of a navigation channel, while others are used primarily in the final refinement phases of the project. For example, the use of underwater sills (see Section 5.2.6) was never contemplated in the early design of the Missouri River navigation channel. The use of such sills was added to the project to provide additional confinement of the channel to realize the final objective, and yet not adversely restrict channel conveyance for high flows.

In the initial design of inland navigation channel the so called "environmental" type modifications presented in Chapter 10 were not originally considered. The vast majority of the inland navigation systems in the U.S. were designed and constructed long before environmental considerations were even recognized as valuable attributes to channel training works. In fact, environmental modifications were implemented over time as design priorities changed. Therefore, modifications to some types of training works, such as dikes, provided environmental enhancement while maintaining their original purpose of contracting the navigation channel and reducing maintenance dredging

4.3 RESISTIVE STRUCTURES

Resistive structures are primarily used to prevent bank erosion and channel migration on the outside of a river bend and to establish or maintain a desired channel alignment. Revetments are usually rock but in the case of the lower Mississippi River, Articulated Concrete Mats (ACM) have been used effectively. Figure 4.2 shows a typical rock revetment.

FIGURE 4.2 Stone revetment protecting a Mississippi River bankline

4.4 CHANNEL ALIGNMENT AND CONTRACTION

The layout of river training structures normally depends on the limits of contraction required to maintain a self-scouring channel of adequate width and depth through the full range of flows to permit continuous navigation. It is desirable to keep the degree of contraction to a minimum so that during flood flows velocities are not too high for safe navigation. Existing channel alignment may require adjustment to allow some design vessel passage. These adjustments can be bendway cutoffs or bend radius modifications.

When setting up the channel alignment, major cutoffs may be necessary; however these should be minimized if possible. Cutoffs are discussed in detail in Section 4.5 below. River systems that are truly self scouring should avoid long straight reaches, and failure to do so will result in costly dredging requirements. This is not always possible, as bankline development, bridges, etc, often restrict the degree of flexibility available to the designer. However, this design element is one of utmost importance for the system of dikes and revetments to perform satisfactorily. The selected structural layout must be compatible with the overall river alignment in order to be successful, and these two elements, alignment and structural layout, must support and complement each other in order to be successful.

Sediment continuity must be maintained when the channel alignment is altered or channel contraction is implemented. The limits of contraction (called channel control lines, rectified channel lines, contraction lines or stabilization lines) are normally determined through a combination of experience, use of model studies, and an analysis of existing river cross sections and sediment data. Analytical models may also be useful in determining the impacts of various channel alignments and contractions on flood heights, velocities, and sediment-carrying characteristics. Through experience and judgment the designer can evaluate various reaches of the stream that maintain adequate depths with natural contractions, and use that information as a basis for determining the required contraction for other reaches. Using model studies, either physical or numerical models, a contraction width can be determined for the problem reach or reaches. Since there is a major change in the channel hydraulics and sediment dynamics, normally a significant amount of modeling is required when a system is converted from an open river condition to a canalized waterway using locks and dams. In this particular case, river training structures are often required in the reaches immediately downstream of the locks and dams to maintain the required channel dimensions without dredging. Additional Contraction Structures (ACS) may also be added to any reach were sediment conveyance is a problem such as where the channel widens or the velocities slow down.

ACS is usually required in a bendway when non-erosive material or revetment for bank stabilization on the outside of the bend is incapable of maintaining the desired, design navigation channel. Revetment alone will hold the concave bank in place; however, in the long-term the natural meandering characteristics will tend to develop chutes through the sandbar opposite the revetment or the crossings will change causing the revetments to be flanked upstream and/or downstream of the crossing. Dikes stabilize the crossings between the pools and the wide sandbars adjacent to the pools along the convex bank. In most cases neither structure alone, dikes or revetment, will accomplish long-term stabilization and maintain the desired navigation channel. Therefore, dikes and revetments work together to stabilize a meandering alluvial river. In the alluvial Lower Mississippi River the revetments are used to stabilize the concave banks adjacent to the deep pools where the banks are most susceptible to bed and bank erosion. Dikes will hold the crossings and stabilize the sandbars, but without revetment, the banks will continue to cave changing the bendway radius and flanking the dikes at the downstream crossing.

In lock and dam systems, ACS will help move the sediment down river during open river flows allowing the system to pass its natural sediment load. An example is John H. Overton Lock and Dam on the Red River which has the Hog Lake additional contraction structures as shown in Figure 4.3

FIGURE 4.3 Hog Lake additional contraction structures

If experience or an in-depth analysis indicates potential increased bank scour in the vicinity due to designed channel improvements, additional modeling and/or monitoring could be performed to address this concern. Numerous measures could be considered due to variables along each portion of the river system, such as evaluating the bankline geomorphic structure (e.g., alluvium or hardpan) and changes in flow patterns (velocity and/or direction) resulting from ACS. Corrective measures could range from modifying project design to include shoreline reinforcement/revetment, as needed or modification to the ACS to periodic surveys for determining lateral channel movement, if any.

4.5 CHANNEL REALIGNMENTS (CUTOFFS)

Channel realignments, frequently referred to as cutoffs, are defined as the realignment of the existing channel by way of a shorter, flatter radius channel across the neck of the bendway. Realignments can be either natural or man-made. Natural realignments result from the continued natural migration and meandering of channel bends. Bends typically migrate laterally and down valley. Realignments occur when the upstream bend migrates at a faster rate than the next downstream bend or the upstream bend continues to migrate and the next downstream bend does not. The rate of migration is a function of both the erodibility of the bank material and the hydraulic forces acting upon the bank. If the upstream bend is comprised of more easily erodible material than the downstream bend and/or if the attack of the flow is greater in the upstream bend, a channel realignment is likely to occur. If the downstream eroding bankline encounters bank material that is not erodible, the bankline ceases to migrate while the bankline in the upstream bend continues to migrate. This tightening of the neck of the bendway, usually results in a channel realignment. Figure 4.4 is a photograph of an impending natural cutoff.

Figure 4.4 Impending Natural Channel Realignment (Cutoff)

Man-made channel realignments are the result of excavating a channel across the neck of a bendway. The primary purposes of these realignments include:

1. Eliminate bends too tight to be safely navigated by commercial tows
2. Reduce flood stages by creating a more efficient channel
3. Provide indirect bank stabilization by realigning the river away from eroding banks

The U.S. Army Corps of Engineers gained valuable experience in the design of channel realignments for navigation during the 1930's on the Mississippi River, during the 1950's and 1960's on the Arkansas River, and during the 1970's, 1980's and 1990's on the Red River in Louisiana. On these rivers, the pilot channel concept of channel realignment was used. This construction procedure includes excavating a channel of smaller section than the desired river section and allowing the natural erosive action of the river to develop the channel to its ultimate dimensions. This method of channel realignment reduces the project cost by significantly reducing channel excavation quantities. However, as pilot channels develop, they introduce additional sediment that must be digested by the river system. The USACE (1980a) recommends that the size, slope, and alignment of the pilot cut should be such that the pilot channel will develop naturally to take most or all of the flow of the stream. The rate of development of a cutoff depends on the erodibility of the material through which the cutoff traverses, size and shape of the pilot cut, length of the cutoff with respect to length of the channel around the bend, and location of the entrance with respect to the alignment of the existing channel. Each of these factors must be considered in the design of pilot channels to insure their development without greatly increasing upstream flood stages or significantly disturbing the stability of adjacent natural river reaches. The length reduction through the pilot channel as well as the size, shape, and alignment of the pilot cut determine the hydraulic factors which impact the rate of development. These hydraulic factors include both the amount of flow entering the pilot channel and the resulting flow patterns. The alignment also impacts the amount of bed load entering the pilot channel from upstream.

Within the waterway reach of the Red River (J. Bennett Johnston Waterway), 36 channel realignments were constructed that shortened the river by almost 50 miles. The pilot channels excavated for these realignments ranged from 80 feet to 200 feet wide. The excavated width was based on the probability of the pilot channel developing. The pilot channels with the greatest probability of development were those with the greatest slope advantage (long old bendway replaced by a short realigned channel) and those that traversed the more easily erodible soils. Of the 36 channel realignments, 33 adequately developed within relatively short time periods. Once the pilot channels were opened, development typically occurred quickly, usually within no more than a couple of highwater events. The three pilot channels that did not develop at an acceptable rate were excavated through backwater deposits of stiff clays. During highwater, the slowly developing pilot channels constricted the channel and significantly increased local channel velocities. These increased channel velocities resulted in the pilot channels being unsafe to navigate. Therefore, all three pilot channels were eventually widened by mechanical dredge.

The development of a pilot channel can be facilitated by constructing a closure structure across the upstream end of the old bendway to force as much of the flow as possible through the realigned channel. On the J. Bennett Johnston Waterway, non-overtopping earthen closure dams were constructed across the upstream end of the old bendways that were being preserved for environmental and recreational purposes. The project design criteria included the preservation of the old bendways that were at least one mile long. For these closure dams, lower stone closures were constructed across the upper end of the old bendway and hydraulic dredge material was pumped between the stone closures to construct a base for the earthen closure dam. The remainder of the closure dam was constructed by land based equipment. The closure dams were constructed to an elevation equal to the post project 100-year frequency flood elevation plus 3 feet. Once constructed, the closure dams were seeded with grass to prevent localized erosion from rainfall impact and runoff. Whenever practical, positive closure was made by tying the closure dam into natural high ground or an existing levee. These non-overtopping closure dams also provide access by local landowners to the islands created by the realignment of the river channel.

The downstream end of the old bendways that include a non-overtopping upstream closure are left open to the river to allow fish migration and recreational access into the old bendway and to allow for an interchange of river water. However, undesirable sediment deposition that threatens to limit or at some locations eliminate river access occurs in the downstream end of the oxbow bendways. The severity of this deposition varies and is dependent upon several factors including the alignment of the old bendway with the realigned channel and the location of the old bendway within the pool. Shallow, narrow low water outlet channels typically develop through the deposition in the downstream end of the oxbows. However, these small channels may not provide adequate year-round access to the oxbows from the main river. Maintaining a clear, reliable connection between the main river and the oxbow is paramount in ensuring long-term access for natural fish restocking, fresh water ingress, and recreational boaters. On the J. Bennett Johnston Waterway, some mechanical dredging has been required to maintain this year-round connection between the oxbow lakes and the river channel. Dredging is not required throughout the oxbow bendway but is limited to the very downstream end. In fact, the length of these access channels varies, but typically ranges from 500 feet to 2000 feet.

On the J. Bennett Johnston Waterway, the bendways less than one mile long were not identified for preservation. For those bendways, a low stone closure was provided across the upstream end of the bendway. The height of the stone closures was dependent upon the location within the navigation pool but was sufficiently high to insure channel control. The stone closures force all the flow during low water periods through the pilot channels. During periods of higher flow, the stone closures are over-topped and suspended sediment laden river flow enters the old bendway. The sediment that enters the old bendway from upstream is deposited within the bendway; therefore, continued sediment deposition over time results in the ultimate filling of these relatively short, old bendway channels.

Excavation of pilot channels with a dredge begins at the downstream end and works toward the upstream end. A plug is left across the pilot channel at the upstream end. On some rivers, these plugs have been left and are eroded when overtopped by higher flow. On the J. Bennett Johnston Waterway, the plugs were removed just prior to constructing the earthen non-overtopping closure dams across the upstream end of the old bendways. Within the realigned channel, revetment, required to stabilize the realigned channel, is provided along the concave bank of the desired alignment. On the J. Bennett Johnston Waterway, trenchfill revetments were used. As the pilot channel developed, the trenchfill revetments (see Section 6.2.2) were undermined thus launching the stone down the eroded bank to stabilize it. Figure 4.5 contains a typical channel realignment plan for the J. Bennett Johnston Waterway.

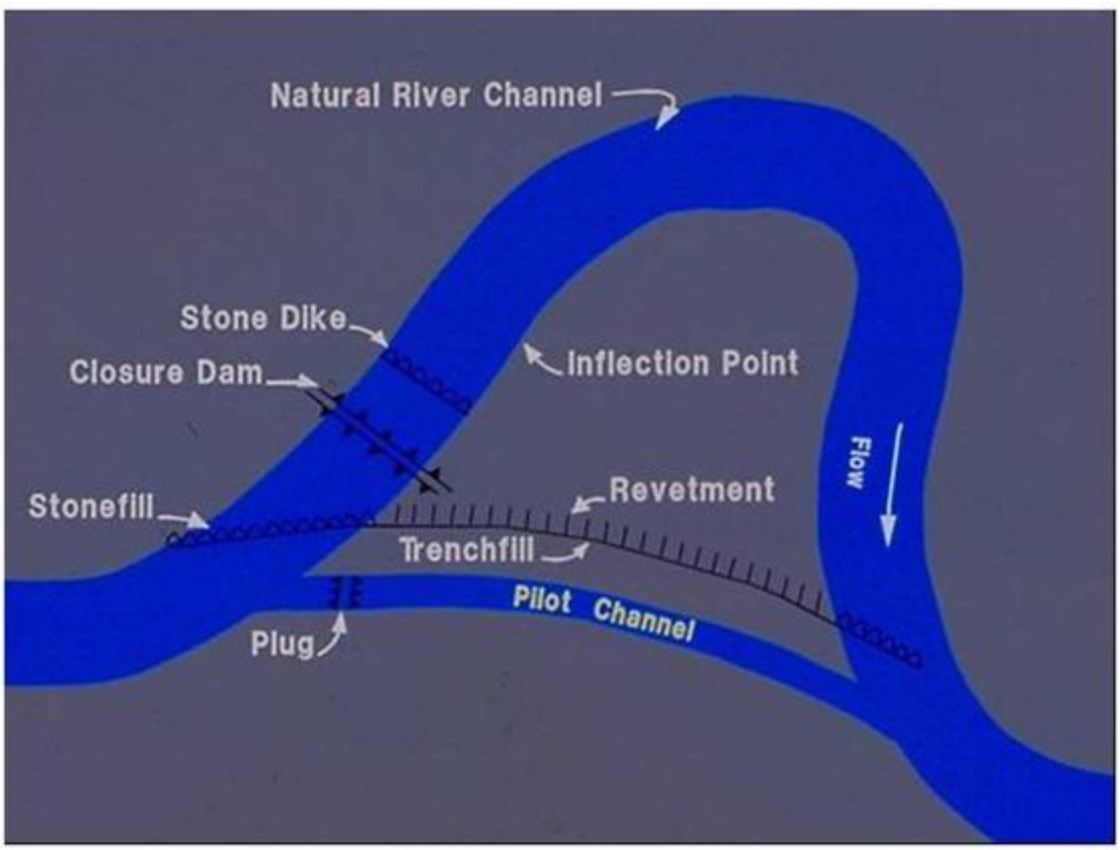

Figure 4.5 Typical Channel Realignment Plan

Phillip Bayou was one of the channel realignments constructed on the J. Bennett Johnston Waterway. That realignment was required to eliminate a sharp bendway which restricted commercial navigation. The realignment replaced a 2.4 mile long bendway with a 1.4 mile long pilot channel. Figure 4.6 identifies the primary components of the Phillip Bayou Realignment. The pilot channel consisted of a trapezoidal section with 1V on 1H side slopes and a bottom width of 80 feet. The bottom grade was excavated to ALWP (Average Low Water Plane) minus 6 feet which resulted in an average elevation throughout the cut of 40 feet, NGVD. Soil borings taken prior to construction indicated the presence of primarily sand, silty sand, and silt overlaid by relative thin layers of both lean and fat clays. The Phillip Bayou pilot channel excavation extended into the more easily erodible soil layers. The Phillip Bayou Realignment also included the construction of a non-overtopping earthen closure dam across the upstream end of the old bendway in order to preserve the old bendway. The construction of this closure dam also facilitated development of the pilot channel by forcing all of the river flow through the pilot channel. A trenchfill revetment was constructed along the concave bank of the ultimate channel alignment to insure the stabilization of the desired navigation channel.

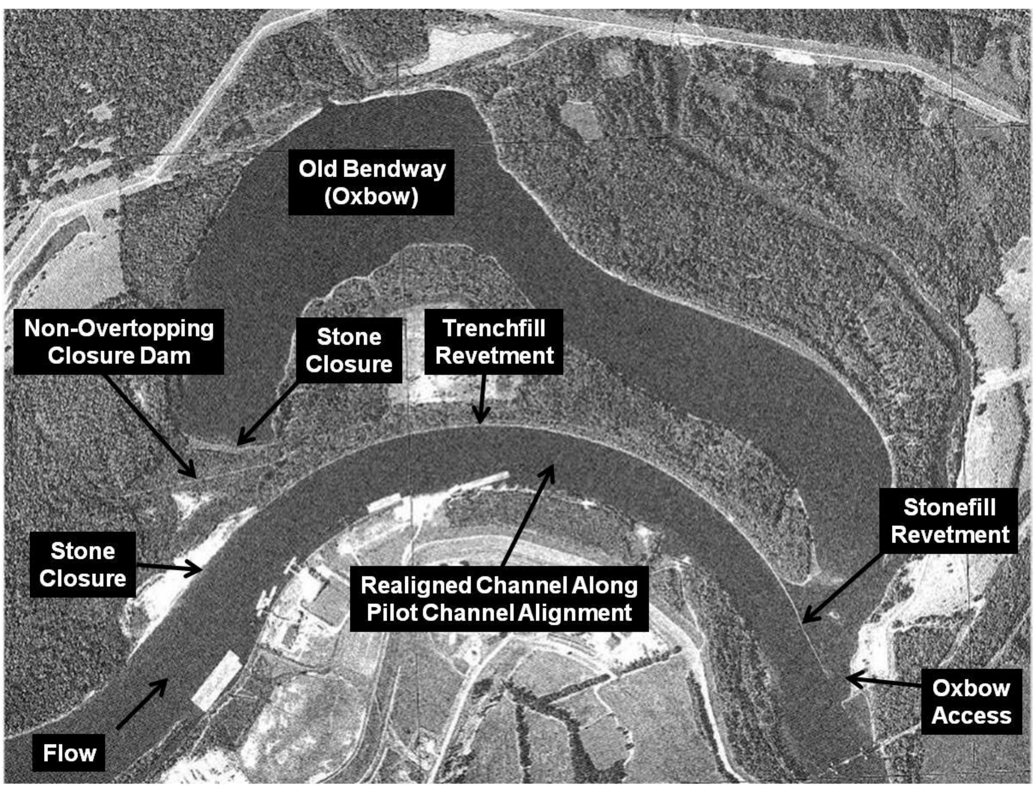

Figure 4.6 Phillip Bayou Realignment Components

Once the plug was removed from the upstream end of the pilot channel on March 6, 1986, the development of the pilot channel was monitored. Cross sections that chronicle the development of the pilot channel are provided in Figure 4.7. These comparative cross sections show a rapid development of the pilot channel. During the first month, the pilot channel scoured and eroded laterally to the trenchfill revetment. During the next three months, the pilot channel continued to scour and widen by eroding its convex bank. At that point, the comparative cross sections show that the bottom grade stabilized but some widening continued. For the four months subsequent to removal of the plug, the hydrograph shows that for about two-thirds of the time, the flow exceeded the average annual discharge.

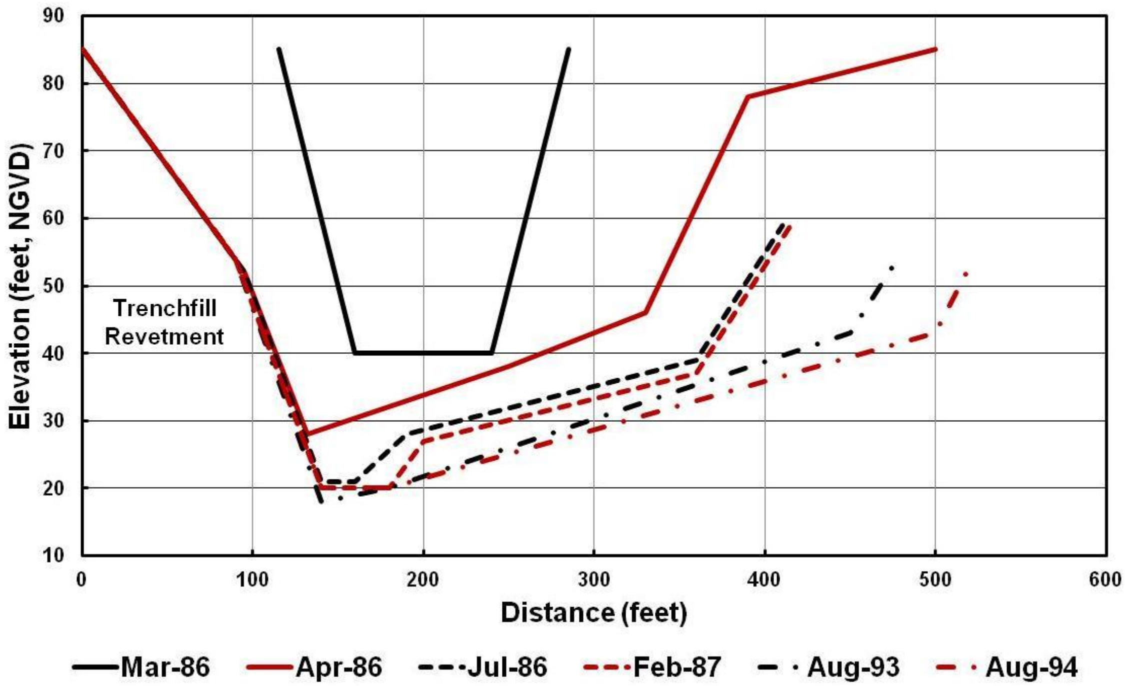

Figure 4.7 Phillip Bayou Realignment Pilot Channel Development Comparative Cross Sections

Construction of the Phillip Bayou Realignment has provided a safe, efficient navigation channel, and the development of the new channel occurred as designed. The reduction in length due to construction of the shorter pilot channel, the high erodibility of the material comprising the bed and banks of the cut, and the four months of higher flows experienced subsequent to opening the pilot channel contributed to the desired development. Also, the construction of the non-overtopping closure dam across the upstream end of the old bendway which forced all the flow through the pilot channel aided in the timely development.

A second channel realignment example on the Red River is Kateland Cutoff. This channel realignment was constructed in 1972 for the purpose of alleviating rapid bank caving along the concave bank of the existing bendway which threatened the integrity of a portion of the Red River levee. Since the bendway was long, the option of constructing a realignment was less expensive than constructing a revetment around the long bend. Construction included the realignment of a 3.1 mile long bendway with a 1.3 mile long pilot channel. The pilot cut consisted of a trapezoidal channel section with 1V on 3H side slopes and a bottom width of 100 feet, except for the extreme downstream 400 feet of the cut which was excavated to a width of 200 feet. Soil borings acquired prior to excavation indicated the presence of primarily silt, fine sand, and silty sand overtopped by 5 to 10 feet thick layers of both lean and fat clays. The pilot channel excavation extended through the clay layers into the more easily erodible soil layers.

The Kateland Cutoff realignment included two design components different than those used on the Phillip Bayou Realignment. At the time the Kateland Cutoff was constructed, the design included leaving a plug across the upstream end of the pilot channel. This plug prevented the lower flows associated with potential sediment deposition from entering the pilot channel. The plug was designed to erode when overtopped by higher flows, Secondly, a non-overtopping closure dam was not constructed across the upstream end of the old bendway resulting in divided flow between the pilot channel and the old bendway. The Kateland Cutoff was opened in early November 1972. Figure 4.8 is an aerial photograph taken almost three months later in late January 1973. Over time, the old bendway filled with sediment leaving only a meander scar where the old bendway channel had once been. Figure 4.9 is an aerial photograph taken in 1987 that shows the fully developed realigned channel and the remaining meander scar.

Subsequent to opening the pilot channel, its development was monitored for approximately five months. During that time, the Red River observed discharges were above the average annual flow approximately 85 percent of the time. During the first three months, the pilot channel scoured between 20 and 25 feet while widening at a rate of approximately 20 feet per month. For the last two months that the pilot channel was monitored, the bed began to aggrade on average of 10 feet across the developing section while widening at an increased rate of approximately 30 feet per month. During the monitor period, Kateland Cutoff developed at an adequate rate to a width of 250 feet at the excavated bottom grade. The significant reduction of the river length from the old bendway to the pilot channel provided the slope advantage necessary to facilitate pilot channel development. The easily erodible silt and sand through which the pilot channel traversed and the high flow experienced during the monitor period also contributed to the desired development.

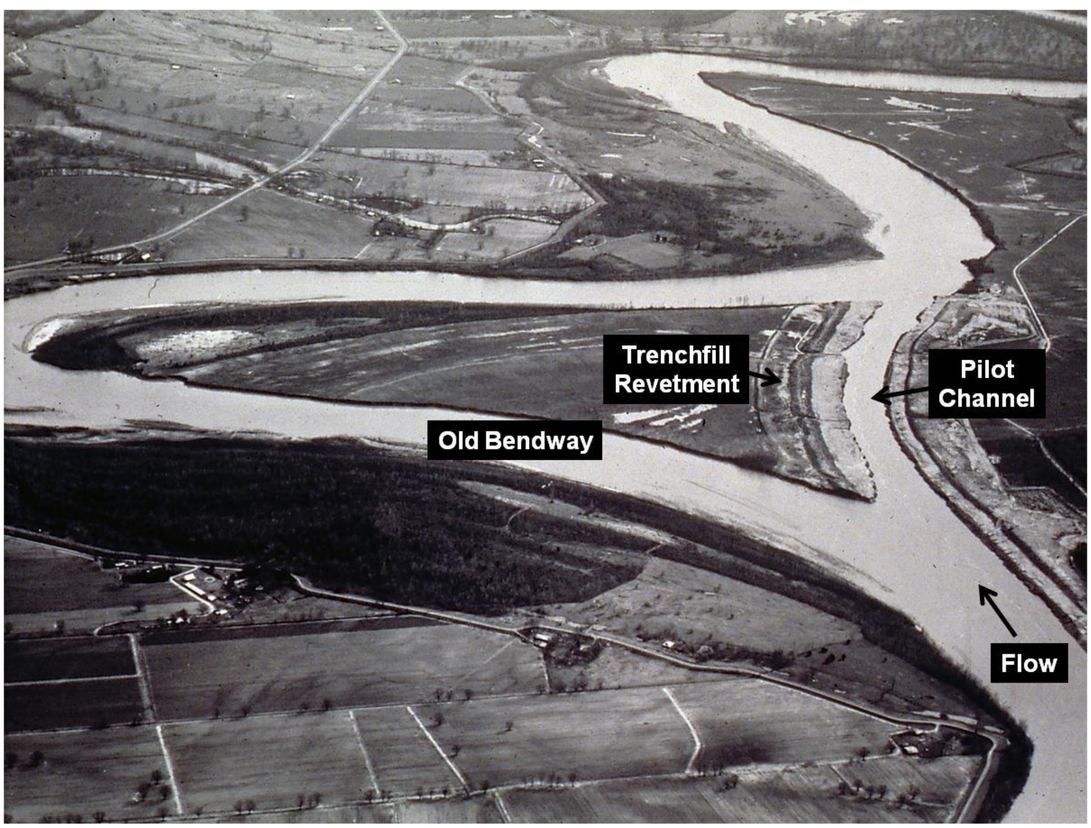

Figure 4.8 Kateland Cutoff Three Months After Construction

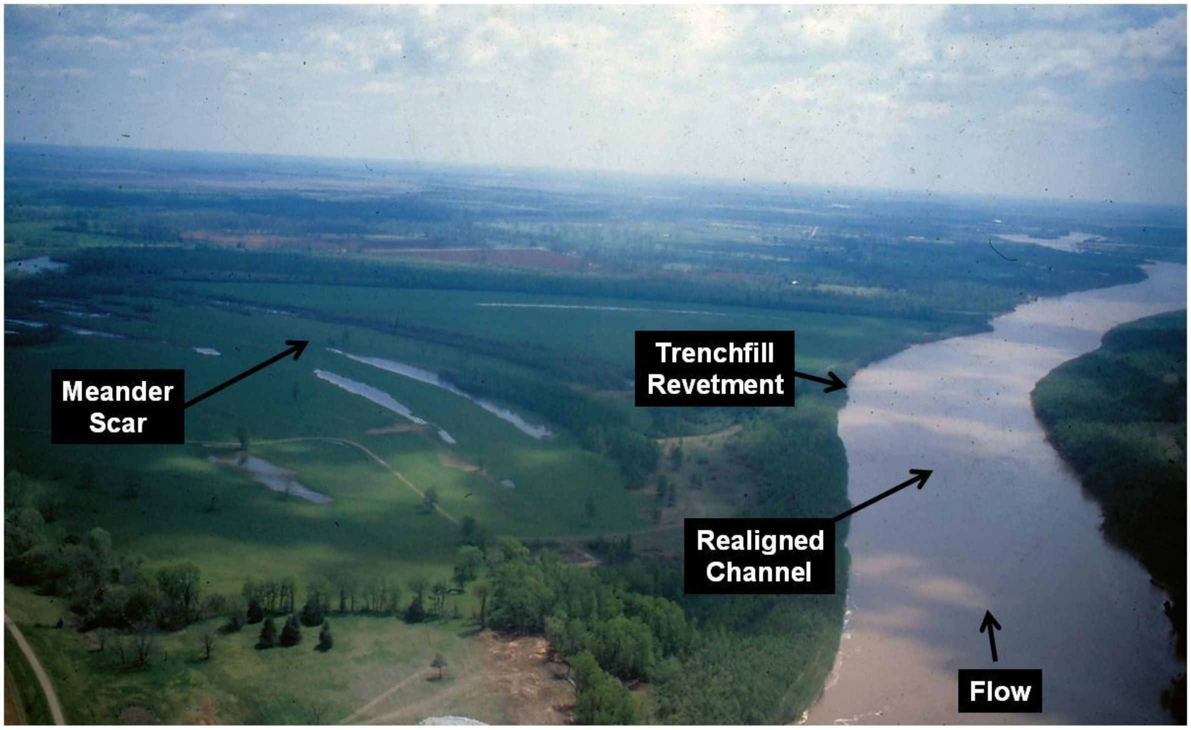

Figure 4.9 Kateland Cutoff Fifteen Years After Construction

Both the Phillip Bayou and Kateland Cutoff pilot channels developed to the desired channel section within an acceptable period of time. The slope advantage gained by realigning natural bendways with shorter cutoffs, the high discharge hydrographs experienced after opening the pilot channels, and the easily erodible material through which the pilot channels traversed contributed to the development. At both realignments, the soils through which the pilot channel was cut and the discharges experienced during the development stage were very similar. Even though the reduction in river length and therefore, the slope advantage was greater for the Kateland Cutoff, the Phillip Bayou pilot channel developed at a faster rate. This accelerated development appears to be the result of the two differences in project design; the removal of the plug across the upstream end of the pilot channel and the construction of a non-overtopping closure dam across the upstream end of the old bendway at Phillip Bayou. This combination resulted in all river flow being forced through the pilot channel, thus taking advantage of the erosive force of the full range of flows. Both the Kateland Cutoff and Phillip Bayou Realignments have functioned as designed by providing a safe, efficient navigation channel.

4.6 CONTRACTION WIDTH

The term contraction width is typically used to describe the amount of contraction that river training structures are designed to provide. The contraction width varies according to the size of the river and the authorized navigation depths. The contraction width is generally thought of as a design guide, rather than a hard and fast rule. On the middle Mississippi River from St.

Louis, Missouri to Cairo, Illinois, a low-water contraction width (usually determined at one-half bank height or less) of 1,500 feet is generally used as the guide. However, in the late 1960's a prototype reach of 14 miles was used to test a 1,200 foot contraction width. The result was that dredging was reduced by 96% and to this day, that trend has continued.

On the lower Mississippi River (downstream from the confluence of the Mississippi and Ohio Rivers), a 2,500 foot (2,200 feet in the Memphis District) contraction width is used as the rule of thumb. On the USACE master plans, the contraction width is delineated with two parallel evenly spaced lines that follow the direction of the river channel. These lines are called the Channel Stabilization Lines. Most dikes typically terminate at these lines. However, in many cases, such as bendways, one of these lines falls along a bankline while the opposite line marks the ends of the dikes on the other side of the river. In channel crossings or straight sections, where there are dikes on both sides of the river, the lines will mark the ends of the dikes on both sides. Historically, river engineers have typically designed river training structures that begin at the bankline and terminate at the contraction line.

There are instances where it has become acceptable to extend training structures beyond the contraction line, if a) additional contraction is needed to resolve a dredging problem or b) a gap is left on the bankline side of the structure to maintain a secondary channel for environmental reasons. The width of this gap can usually be added to the overall contraction width of the channel. On the middle Mississippi River, engineers typically do not extend beyond a 1,200 foot contraction width with any structure. It has been found that these contraction widths provide adequate depth for navigation during periods of low flow and effectively increase the rivers conveyance during high flows. The theory and analysis behind contraction widths is presented in the Appendix B paper *Sediment Management on the Mississippi* by Max Lamb and Loyde T. Ethridge.

4.7 GENERAL CHANNEL PLAN

After the contracted channel width has been determined, it is necessary to lay out the desired channel alignment within the existing project limits. In some cases a major channel realignment including a cutoff may be required to meet the project requirements, but in most cases the contracted channel can be laid out within the existing channel top banks (Figure 4.10). The contracted channel width is established on the map with the channel control lines providing the designer with the "blueprint" for the desired channel alignment. Although minor adjustments of the alignment may be necessary in the future to account for changes in the project, this map serves as a master plan for the project and the eventual goal for design purposes. The most important point to be made relative to the contracted channel layout and establishment of the channel control lines is that every effort should be made to follow the natural river tendencies and to avoid a "forced" channel alignment. By following the natural pool-crossing-pool sequences and sinuosity and providing smooth transitions between bends and adequate crossing lengths between pools, maintenance dredging costs will be minimized or entirely eliminated. The design should ensure that crossing lengths between bends provides smooth transition form the upstream bend and into the downstream bend (USACE, 1969a). When crossings are overly long, the stream may tend to meander and encourage the development of middle or alternate bars within the channel (Figure 4.11). Experience has shown that river reaches that have been over contracted, poorly aligned, or established against the natural tendencies of the particular stream tend to have high maintenance costs, poor navigation conditions, or difficulty in maintaining adequate channel dimensions (USACE, 1969a). The designer should make every effort to ensure that the final layout is compatible with the stream's natural tendencies and that realignments fit within those tendencies also. At a given cross section, dike length is the major parameter that controls the amount of channel contraction, while the dike height and crest profile impact the stability of the dike system. Providing dike lengths to the desired channel control lines is adequate for initial construction; however, once the stream responds to the dikes, modifications and adjustments may be required.

There is another important controlling factor that needs to be considered in the layout of the navigation channel – the design tow and the way tows will move through the channel being designed. Franco and Shows (1982) conducted a comprehensive study to determine channels widths in bendways and the length of channel (the crossing) between bendways required for safe and efficient towboat navigation. The results of that study are also presented in Chapter 5 of McCartney et al, 1998. The study provides the designer with information for bendways with uniform or compound curvature and with regular and irregular bank lines. Design guidance is provided for tows moving upstream and downstream and covers six (6) different commonly configured tows. It is strongly suggested that the designer refer to one or both of these references when laying out the navigation channel limits during the design process.

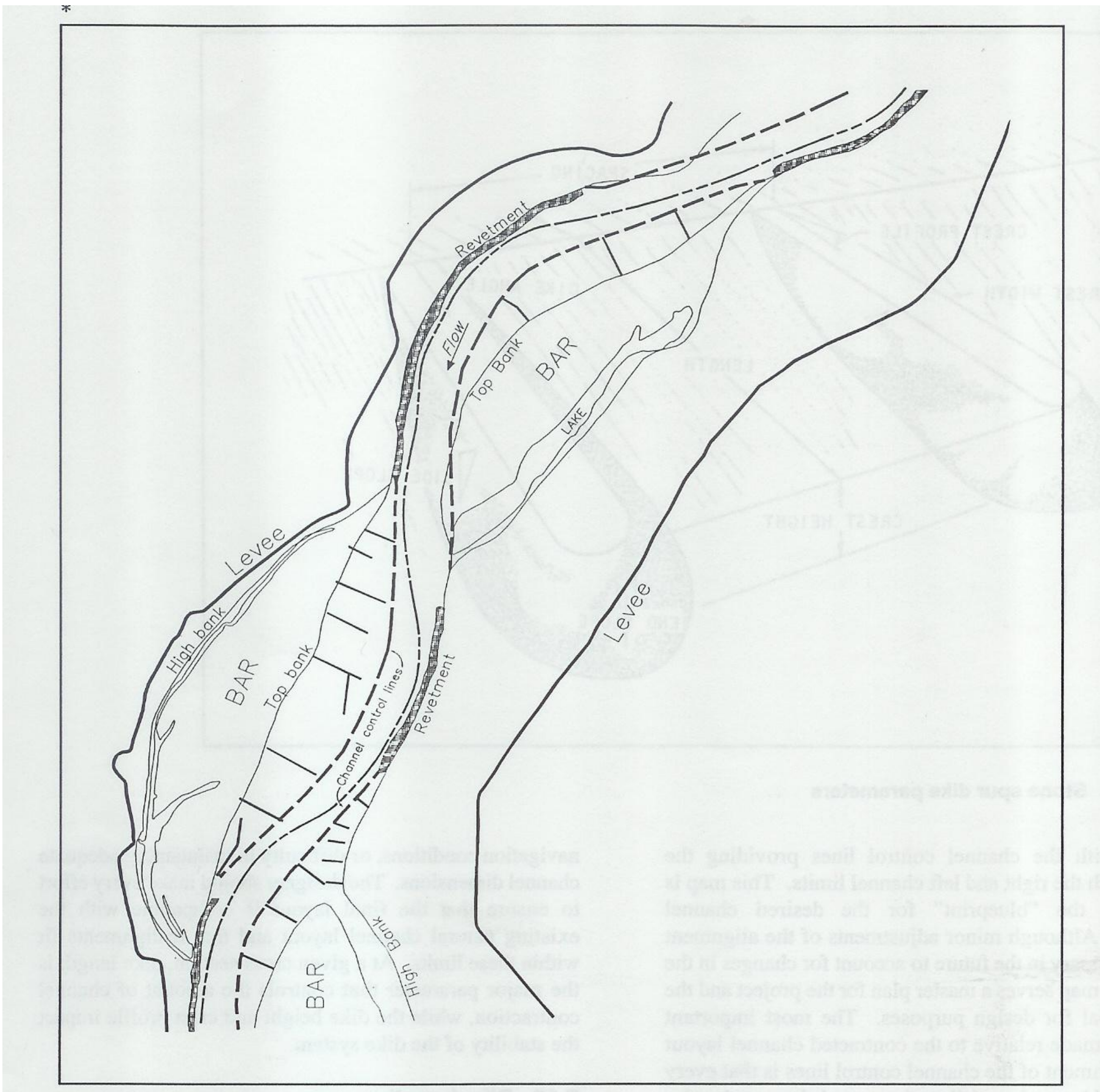

FIGURE 4.10. Example of layout for channel control lines

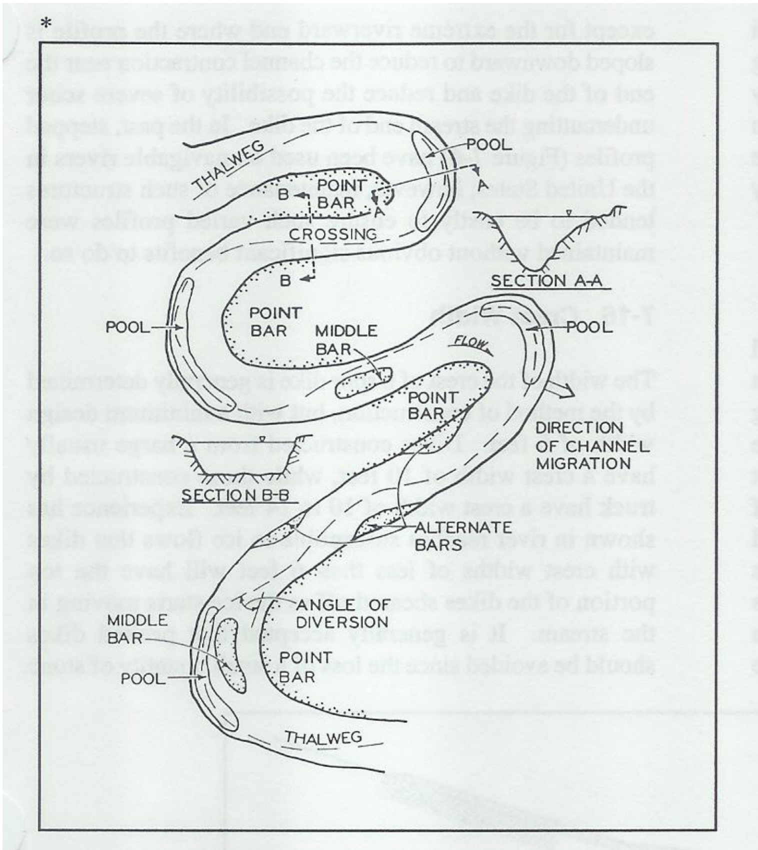

FIGURE 4.11. Channel planform

When considering the channel design and layout, it is imperative that the designer realize that the design needs to be flexible and not a hard, firm and fixed-in-place approach. After the initial design is completed and construction has been initiated in the river, it may soon become apparent that the originally proposed layout is not exactly what is needed. This may be due to river flow conditions following training works construction or other factors. Regardless of the reason, it may become necessary to modify the original layout to achieve the desired channel alignment.

4.8 LOW WATER REFERENCE PLANE

The contraction width is designed such that the result of the structures built to this width will provide a usable navigation channel width and depth at low water. The definition of low water in open river projects varies, e.g., on the Mississippi River, it is referred to as the Low Water Reference Plane (LWRP); on the Missouri River, it is termed the Construction Reference Plane (CRP); on the Columbia River, it is called the Columbia River Datum (CRD); and on some other systems it is called the Annual Low-Water Plane (ALWP). In canalized waterways the reference plane is normally called the Low Operating Level, referencing the minimum upper pool elevation upstream of the dam and lower minimum pool elevation downstream of the dam; this is sometimes informally – and technically inaccurately – referred to as flat pool.

The Mississippi River LWRP is a hydraulic reference plane established from long term observations of the river's stage, discharge rates, and flow duration periods. The low water profile was developed about the 97-percent flow duration line. The elevation of the 2007 LWRP on the middle Mississippi River drops gradually throughout the course of the river, however, some anomalies in the profile are present in places (particularly in areas containing rock bottoms). The average gradient is approximately 0.5 feet per river mile. The ever-changing river bottom influences the 2007 LWRP, so engineers use these elevations as a guide rather than a rule. Continual changes in the stage-discharge relationship will influence the theoretical flow line for the 2007 LWRP and re-evaluation will be necessary. All construction, improvements, and dredging along the lower river are performed relative to the 2007 LWRP at a particular point.

CHAPTER 5

DIKES

5.1 STONE SPUR DIKE DESIGN PARAMETERS

A dike is defined as a structure placed approximately perpendicular to the bank line to prevent bank erosion and protect structures along the bank, realign a river reach, constrict a channel to increase depth, cutoff side channels and chutes, and concentrate the flow. The design of dikes must consider parameters such as channel alignment, contraction, dike length, dike height, crest profile, crest width, side slopes, end slopes, dike angle, dike spacing, stone size, bank paving, method of construction and water depth (see Figure 5.1). Stone is currently the first choice for dike construction but timber dikes often have a cost advantage in deep water applications. In some cases, a combination of stone and timber construction may be used to take advantage of the attributes of each type of structure/material type with stone along the lower cross-section and bank line to address scour and timber piles along the upper cross-section to provide increased flow permeability higher in the water column

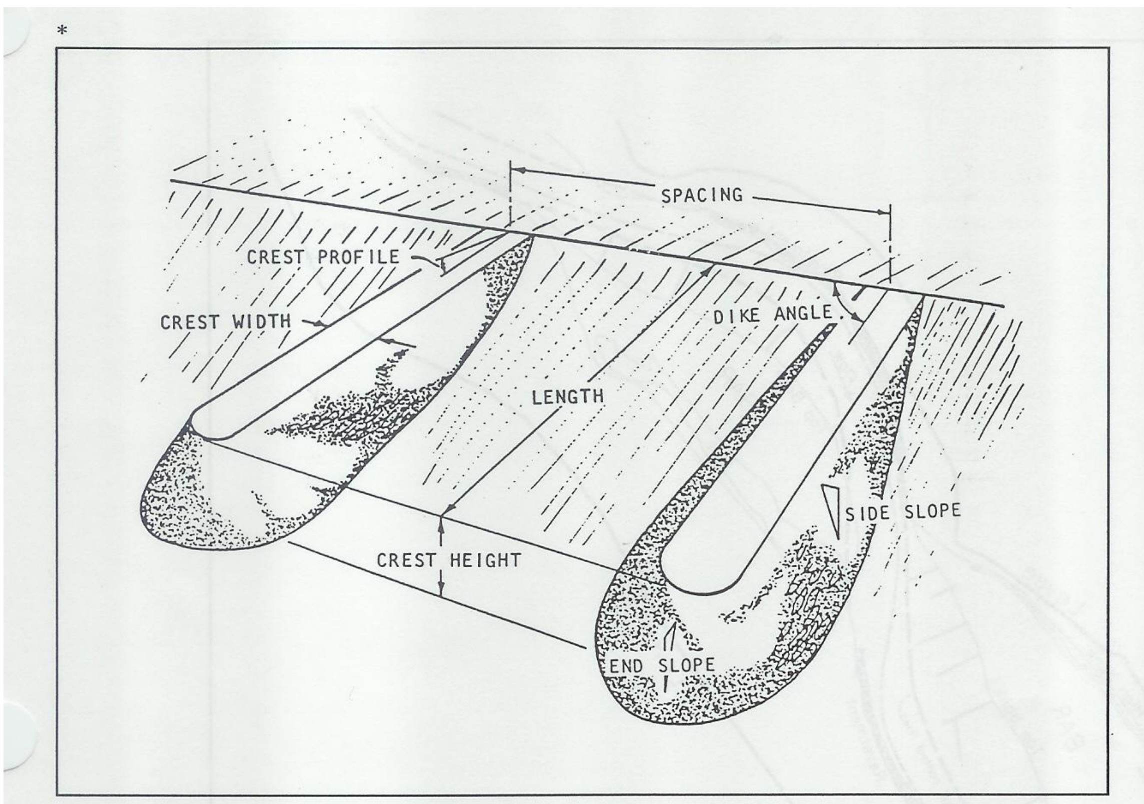

FIGURE 5.1 Stone dike parameters (USACE 1997)

5.1.1 DIKE LENGTH

The length of dikes is controlled by the desired contracted channel width, since dikes extend from the bank to the channel control line on the same side of the river. There are instances where dikes may have lengths that infringe on the channel control lines, but these are special situations where some slight added contraction is required due to site specific conditions and/or where the portion of the dike riverward of the channel control line was at a significantly lower elevation than the main portion of the dike. Providing dike lengths to the desired channel control lines is adequate for initial construction; however, once the stream has responded to the dikes, modifications and adjustments may be required.

5.1.2 DIKE HEIGHT

The height or top elevation of dikes is normally associated with the reference plane associated with the project (see Section 4.7). The elevation of the dikes relative to the water surface can have an important bearing on the structure's performance, its impact on the stream, and its impact on the areas within the dike field. On open river portions of the Mississippi River the top elevation of dikes typically varies from about 10 to 18 feet above the LWRP. Normally this puts the height of the dikes approximately at the midbank elevation. On the Missouri River, an analytical procedure is used that assists in selecting the proper dike height. The procedure provides a design relative to a flow and/or stage-duration curve of the river and particular reach. Specifically what is being addressed is the percent of time that a given stage or discharge is equaled or exceeded. This kind of information assists in determining the chances of dikes being overtopped when constructed to various elevations, and provides a methodology for selecting the height to which certain structures should be constructed and maintained. Generally speaking, dikes constructed to lower elevations will require more maintenance than dikes constructed to higher elevations. Experience on the Missouri River has shown that dikes that are seldom overtopped will usually develop a significantly different depositional pattern downstream of the dike than those that are frequently overtopped. Areas downstream from dikes that are frequently, but not continuously, overtopped will develop a shoaling pattern within the dike field that is almost uniformly at or slightly above the average water surface elevation. Dikes that are seldom overtopped will form a depositional pattern immediately downstream and landward of the stream end of the dike that often leaves an open water area between the deposit and the original bank line (USACE, 1969a). On canalized projects the top elevation of dikes is referenced to the Low Operating Level, the minimum regulated pool elevation or some similar reference. In some pools the top elevation of the dikes is about 2 or 3 feet above the pool elevation regardless of the location in the upper or lower pool, to help provide visibility of the structures to river pilots. That elevation is a minimum to ensure that pilots and recreational boaters can see the dikes at most times and be aware of the existence of river training structures in that location. In many other pools, the top elevation of dikes is about 2 to 3 feet below the pool elevation; this helps to provide flow in back and side channels during low water periods. The tops of these dikes are close enough to the water surface and provide adequate visual indicators to river pilots from the water surface disturbance. In some cases it may be necessary to enhance the visibility of pile dikes for navigation and avoidance purposes. The application of marker piles midway along the riverward span of long dikes and use of king-piles at the riverward endpoint of dikes can improve the visible silhouette of pile dikes during periods of high river stage. The tops of markers piles and king-piles should be 5 to 10 ft above the ordinary high water stage. King-piles can be used as structural elements for attaching dayboards and other navigation aids. In addition, U.S. Coast Guard navigation buoys typically mark dike fields.

5.1.3 ADJACENT DIKE HEIGHTS

The relationship between the height of adjacent dikes in a system, three dikes or more, is also of importance. Some dike systems are constructed such that all dikes are set at the same elevation relative to the particular reference plane in use. In certain applications a stepped-down dike system (dike elevations decreasing moving downstream) promote accumulation of bed material within the dike field and provide for a continuous navigation channel adjacent to the dike field. In some of the dike systems on the Mississippi River a stepped-up dike system (dike elevations increasing moving downstream) has been used to follow the tendency of the bars built naturally by the river. The stepped-down system appears to be the more preferred with dike elevations decreasing by 1 foot from the dike immediately upstream (USACE, 1969a).

5.1.4 CREST PROFILE

The crest profile of dikes, most often used, is level from the bank to the stream end; however, variations to this parameter may be preferred at times. A crest profile sloping down from the bank to stream end is useful if a wide range of river stages are encountered, if a decrease in the amount of contraction is advantageous as river stages increase, or if some erosion of the fill material within the dike field downstream of individual dikes is acceptable. In such cases the total drop in elevation over the length of the dike is about 5 feet (USACE, 1997). Other applications that merit consideration maintain a level crest profile over the length of the dike except for the extreme riverward end where the profile is sloped downward to reduce the channel contraction near the end of the dike and reduce the possibility of severe scour undercutting the stream end of the dike. In the past, stepped profiles (Figure 5.2) have been used on navigable rivers in the United States; however, maintenance of such structures tended to be costly to ensure such varied profiles were maintained without obvious significant benefits to do so (USACE, 1997). Dikes with reverse slope (crest profile sloping up from the bank toward the navigation channel) are being tested on the middle Mississippi River. The objective is to create off-channel habitat adjacent to the main navigation channel while increasing depths in the main channel. Other crest profile variations include notching for environmental purposes (see Section 10.2).

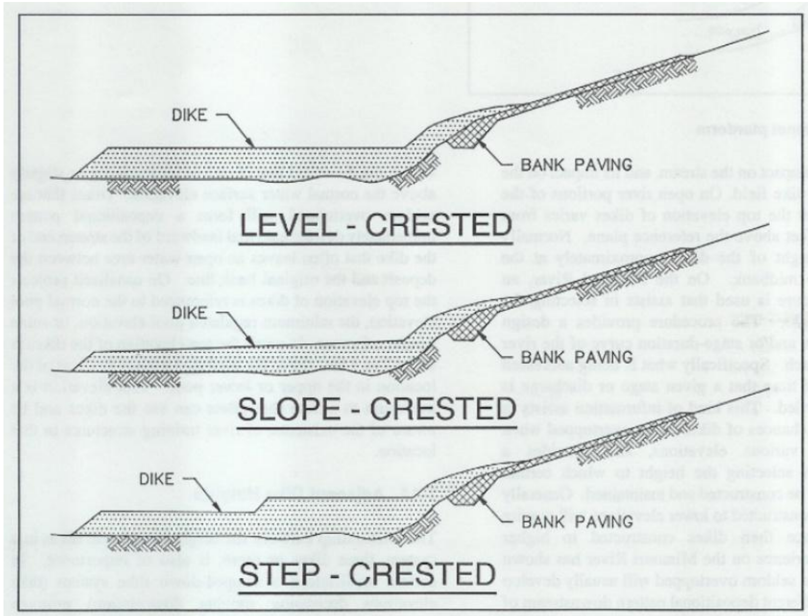

FIGURE 5.2 Stone dike crest profiles

5.1.5 CREST WIDTH

The width of the crest of a stone dike is generally determined by the method of construction, but with a minimum design width of 5 feet. Dikes constructed from a barge usually have a crest width of 6 to 10 feet, while those constructed by truck have a crest width of 10 to 14 feet to accommodate movement of the truck/backhoes and other equipment on the dike structure. Experience has shown in river reaches susceptible to ice flows that dikes with crest widths of less than 6 feet may have the top portion of the dikes sheared off as the ice starts moving in the stream. It is generally accepted that peaked dikes should be avoided since the loss of a small quantity of stone will produce a gap in the dike. This gap could cause scour and possible breaching of the dike with the dike becoming separated from the bank end of the dike. One other method for determining dike crest width is to design the dikes based on the stone size used and the height of the dike. In this case the crest width is allowed to vary so long as the minimum width of 5 feet is maintained. Summarizing, there is some variation in the crest widths used for dikes, but virtually all dikes fit within the range of 5 to 20 feet with the majority of dikes constructed with a crest width of 5 to 10 feet. A rule of thumb on the lower Mississippi River is to maintain a minimum of 10 tons of stone per lineal foot of the dike. This provides an adequate section to minimize the risk of failure. In some cases the most upstream dike in a dike field may require a wider crest because this structure will be subjected to the most flow, debris and ice.

5.1.6 SIDE SLOPES

The side slopes (upstream and downstream faces) of dikes usually are maintained on the natural angle of repose of the stone used to construct the dikes. Although this angle varies somewhat depending on the particular stone used, the angle is about 40 degrees, which produces a slope of 1V on 1.25H. Normally the side slopes used on the designs of stone dikes, including computation of required stone quantities, is 1V on 1.25H or 1V on1.5H with the difference being a function of the particular stone, the dike height, and the velocities and depths of water that stone has to fall through during construction.

5.1.7 END SLOPES

Although the slope of the stream end of a stone dike can be as steep as the natural angle of repose (40 degrees or 1V on 1.25H), it is advisable to construct the dike with a somewhat flatter end slope to form a bull-nose for the stone head. The stream end of the dike is the point of contraction and is susceptible to the most bed scour as a result of the streamflow moving around the end of the dike. The steeper the slope on the stream end the greater the chance for loss of stone as end scour occurs. During the design it should be considered how much of the dike length can be lost due to launching of the stone at the stream end and still maintain the effectiveness of the dike (Franco, 1967). Often an end slope of 1V on 5H is used where significant scour is anticipated, but in some unusual and severe scour circumstances an end slope as flat as 1V on 10H has been used. If a natural angle of repose for the dike end is used in the design, the designer can anticipate that a certain amount of stone will be launched into the scour. A slight overbuild may be necessary to accommodate the launched stone which will naturally armor the scour hole and prevent additional loss of the stream end of the dike.

5.1.8 DIKE ANGLE

The angle that a dike makes with the river bank is an important factor in the location and amount of scour that occurs at the stream end of the dike and the location of the channel that develops adjacent to the dike. Experience has shown that as flow crosses a dike it tends to turn perpendicular to the dike; therefore, in most applications dikes are constructed normal to the adjacent bank line or flow. It is obvious that the longer the dike the more expensive it is to build and the shortest distance between the bank and the channel stabilization line is a perpendicular line. Historically, dikes were constructed normal to the adjacent bank line or angled slightly downstream. When dikes are angled upstream the scour at the stream end of the dike will be greater and the channel will be farther from the dikes than systems that are normal or angled downstream. For dikes angled downstream, when the dike is overtopped for long durations, the flow is directed toward the downstream bank line and a blowout of the bank and/or dike root can occur. When use of a downstream angled dike is considered, the designer should use it in conjunction with significant bank revetment to mitigate the potential for downstream bank scour.

5.1.9 DIKE SPACING

The spacing of dikes within a system should be great enough that the least number of dikes are built while still maintaining the effectiveness of the system. If the spacing is too great the channel will tend to meander between the individual dikes. If the spacing is too small the system effectiveness will be equal to that of the ideal spacing; however, such a system will have a greater initial cost without greater benefits when compared to the ideal system. For this reason, structure length and spacing are not considered independent parameters, as a longer structure will generally indicate that the structure spacing can also be increased. A spacing of two-thirds of the length of the upstream dike is probably the minimum spacing that is effective, although it is obvious that as the spacing decreases between dikes the cost of the system will increase. On streams with dike lengths of about 1,000 feet, spacings of 1-1/2 to 2-1/2 times the length of the upstream dike have been used. However, on larger waterways, such as the lower Mississippi River, which have extremely long dikes to reach the channel control line, this guidance would provide an undesirable spacing. In such cases a maximum spacing of 3,000 to 4,000 feet is normally used. Experience on the Missouri River indicates dike length is seldom uniform throughout a river reach, but that the dikes are spaced such that the flow passing around and downstream from the stream end of the structure intersects the next dike prior to intersecting the bank line. The rule of thumb used by the Missouri River designers for dikes on the inside of bends is a spacing of 2 to 2-1/2 times the structure length. Another method used in the past on the Missouri River was to assume that the flow expanded on a ratio of 5 to 1 in the longitudinal direction from the tangent of the flow line off the stream end of the upstream dike with the next downstream dike placed slightly upstream of the intersection of this theoretical expansion line and the bank line.

5.1.10 DIKE BANKHEADS.

Dike bankheads are required to prevent erosion of the bankline that could result in flanking of dikes. The two primary bankhead techniques include; 1. excavating a trench into the bank and filling it with stone, basically extending the dike into the bank and 2. paving the bank with stone downstream of the point where the dike intersects the bank and possibly paving upstream a lesser distance. It should be noted that in certain situations both methods have been used on a dike to protect the dike from flanking. Trenching into the bank is called a root. For dikes in straight reaches, general guidance includes extending the dike root into the bank a minimum distance equal to the bank height. A more conservative approach includes extending the root into the bank a distance equal to the bank height plus the depth of local scour holes in adjacent areas. If eroded "eddy pockets" downstream of existing protrusions into the channel are observed, the root should extend into the bank a distance at least as long as the maximum landward extent of those pockets. For areas of severe erosion, such as in bends, roots should be longer. Roots of as much as 300 feet have been used on the Mississippi River and as little as 10 feet on small streams (Biedenharn, et al, 1997). Figures 5.3 and 5.4 are drawings of two root options used on the Red River Waterway (USACE, 1972). The option used is based on the height of the dike in relation to the height of the bank. Figure 5.3 is the option for dikes that are high compared to the top bank and Figure 5.4 is the option for dikes that are low compared to top bank. There are various designs for stone roots. The primary goal in any design is to make sure that the root extends far enough into the bank and has a sufficient amount of stone to prevent flanking if scour does occur. Typical root design on the Mississippi River in the Vicksburg District, USACE, includes providing a trench that is 12 feet wide with 1V on 1.5H side slopes. The depth of the trench is typically 12 feet below top bank for the riverward most 200 feet and transitions to 7.5 feet below top bank for the landward most 100 feet. Figure 5.5 is a photo of a Mississippi River excavated trench prior to stone placement. Figure 5.6 is a photo of a root after the stone has been placed. For roots that do not extend as high as top bank, the stone in the trench is typically covered with fill material to complete the root construction.

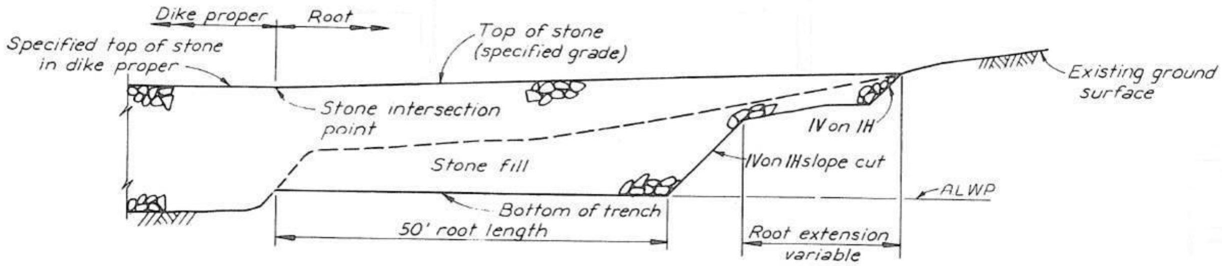

Figure 5.3 Stone Root For Dikes That Are High Compared To Top Bank

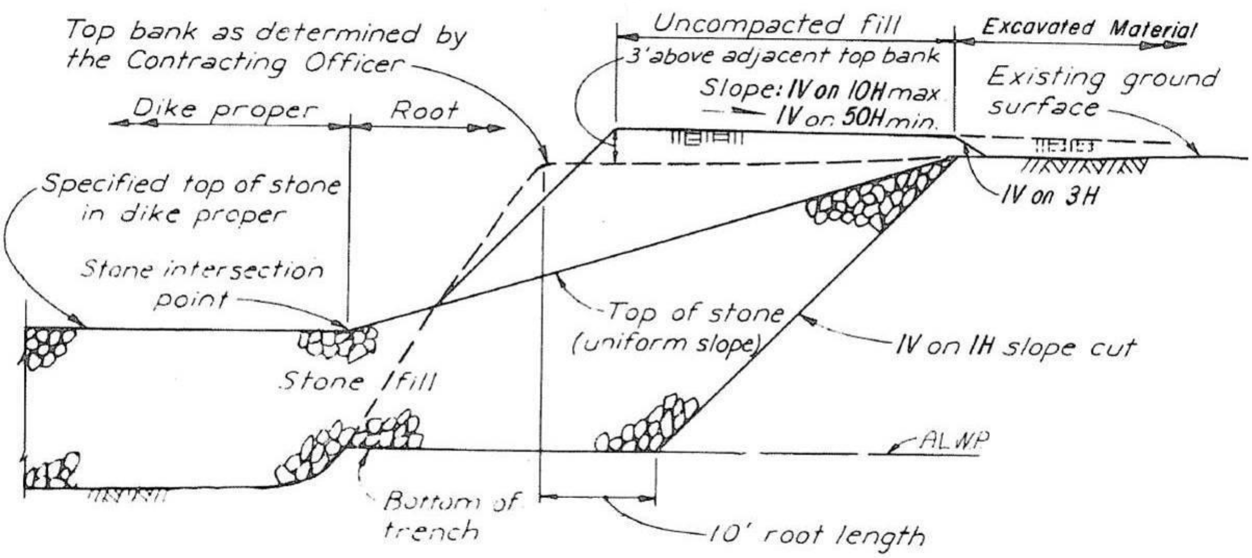

FIGURE 5.4 Stone Root For Dikes That Are Low Compared To Top Bank

FIGURE 5.5 Excavated Trench For Stone Root On The Mississippi River

FIGURE 5.6 Stone Placed In Stone Root On The Mississippi River

The second bankhead option is paving the bank with stone. The general rule is to pave the bank downstream of the dike a distance equal to a multiple of bank height or equal to a multiple of bank height plus scour allowance. For average conditions, a multiple of 3 is adequate. For example, if the bank height is 20 feet and expected scour is 5 feet, the bank paving should extend (20 + 5) x 3 = 75 feet downstream of the dike. Upstream paving is optional but if it is provided, the distance need not exceed bank height (Biedenharn, et al, 1997). In both instances, the bank must be cleared of all vegetation and graded to a stable slope. Figure 5.7 is a photo of bank paving on the upper Mississippi River and Figure 5.8 is a photo of bank paving on the lower Mississippi River. In Figure 5.7, flow is from left to right and bank paving is provided both upstream and downstream of the dike but to a lesser extent upstream. In Figure 5.8, flow is from the right to the left and bank paving is only provided downstream of the dike.

FIGURE 5.7 Dike Bank Paving On The Upper Mississippi River

FIGURE 5.8 Dike Bank Paving On The Lower Mississippi River

5.1.11 SCOUR PROTECTION FOR DIKES

One of the primary causes of dike damage that can lead to structural failure is local scour around the end and adjacent to dikes. Alternative approaches as provided by Biedenharn, et.al. (1997) for reducing or eliminating this scour include:

- Add a stone apron on the stream bed under and adjacent to dikes. The thickness of the apron should be substantial enough to protect against anticipated scour.
- Place additional stone at the end and on the side slopes of dikes. When additional stone is provided on the end and side slopes, a sufficient quantity of launch stone should be added to protect against anticipated scour.
- Add structural features such as L-Head or T-Head to the end of dikes to move scour away from the dike proper. When using L-Heads or T-Heads on the end of dikes, scour will still occur but will occur off the end of the L-Head or T-Head instead of immediately off the end of the dike. This reduces the likelihood of damage to the primary dike section.
- Use hydraulically smooth designs for the end of impermeable dikes and round structural members for permeable dikes. The Federal Highway Administration (1985) suggested that dike heads or tips should be as smooth and rounded as possible to help minimize local scour, flow concentration, and flow deflection.

In order to reduce scour during construction, dike construction should begin at the bankline and progress in a riverward direction. For stone dikes on the Mississippi River, the Vicksburg District constructs dikes in lifts. The first lift includes a minimum thickness of 4 feet. This lift serves as the base and helps prevent bed scour as the dike is constructed.

5.2 OTHER TYPES OF STONE DIKES

The above section provides dike design parameters for spur dikes generally constructed approximately perpendicular to the bankline or direction of flow. This section presents additional types of stone dikes which are constructed similar to stone spur dikes with somewhat of a modified purpose to establish a stable channel alignment.

5.2.1 LONGITUDINAL DIKES

Longitudinal dikes are continuous structures extending from the bank downstream generally parallel to the alignment of the channel being developed. On the Red and Arkansas Rivers, longitudinal dikes are called stonefill or timber pile revetments depending on the type of construction material. Properly designed longitudinal dikes are the most effective type of structure in developing a stable channel since these structures are basically a false bank line; however, these structures are the most expensive to construct due to their long length and required tie-in or baffle dikes. Longitudinal dikes can be used to reduce the curvature of sharp bends and to provide transitions with little resistance or disturbance to flow. However, once in place, it is difficult and expensive to change the alignment of the dike. Figure 5.9 shows the application of a longitudinal dike on the Arkansas River. It should be noted that the tie-in dikes landward of the longitudinal dike add stability to the entire structure. These tie-in (tie-back on the lower Mississippi and Red River) dikes can also be modified using notches or openings to enhance the habitat and maintain open water areas on the back side of the longitudinal dike.

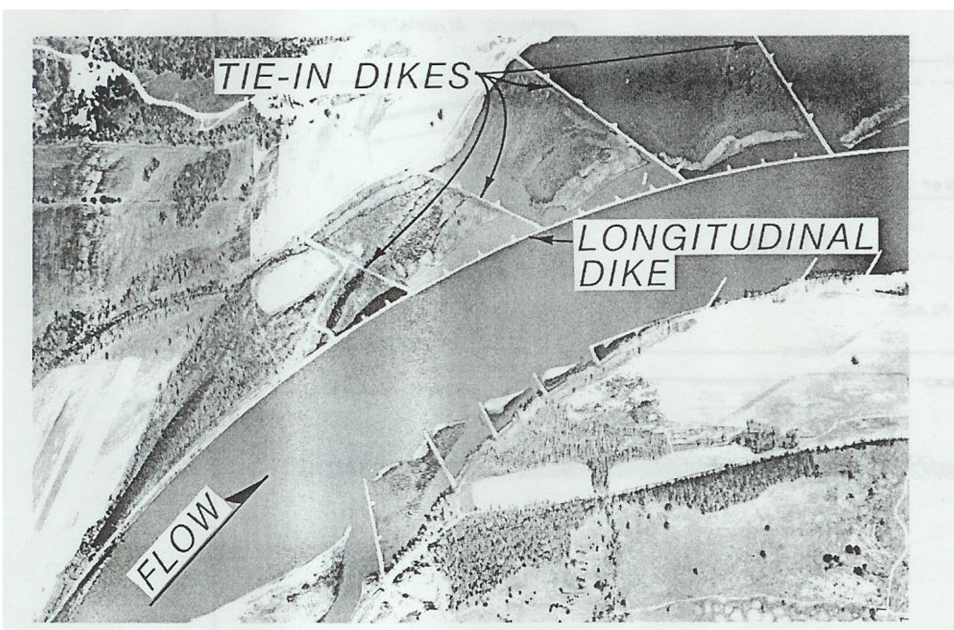

FIGURE 5.9 Longitudinal dike on Arkansas River

5.2.2 VANE DIKES

Dikes placed in the form of a series of vanes have proved effective as a means of controlling channel development and sediment movement under certain conditions (Figure 5.10).

These dikes consist of segments of dikes located riverward from the existing bank with gaps between the dikes. The length of the gaps between the dikes is usually about 50 to 60 percent of the length of each vane. Usually all of the vanes in a system are of equal length. The dikes are placed at a slight angle to the direction of flow, about 10 to 15 degrees, with the downstream end of the dike farther riverward than the upstream end. The system should be placed in an area where there is or will be movement of sediment. These dikes have been used on the major navigable rivers in the United States as independent systems or in conjunction with dike systems. Vane dikes are often less expensive than conventional dikes since they can be placed in relatively shallow water aligned generally parallel to the channel control line and produce little disturbance to the streamflow and are often constructed using a floating plant to place the stone. On some of the vane dike systems that have been in place for many years some of the vanes have been connected to the bank line with a spur dike creating an L-head dike. This modification is often undertaken after significant shoaling of material has taken place between the vanes and the area landward of the dikes (Figure 5.11). In some conditions, vane dikes are not always the best solution. On the lower Mississippi River the vane dikes were not effective in accumulating sediment and were converted to L-head dikes with a tie-back to the river bank.

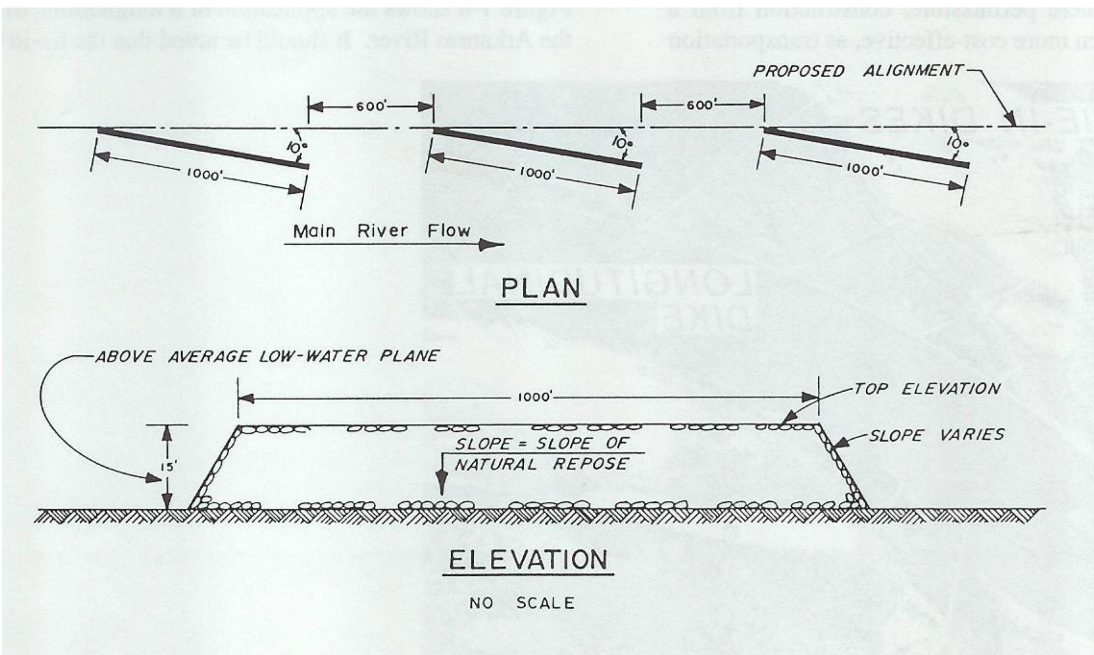

FIGURE 5.10 Vane dike layout

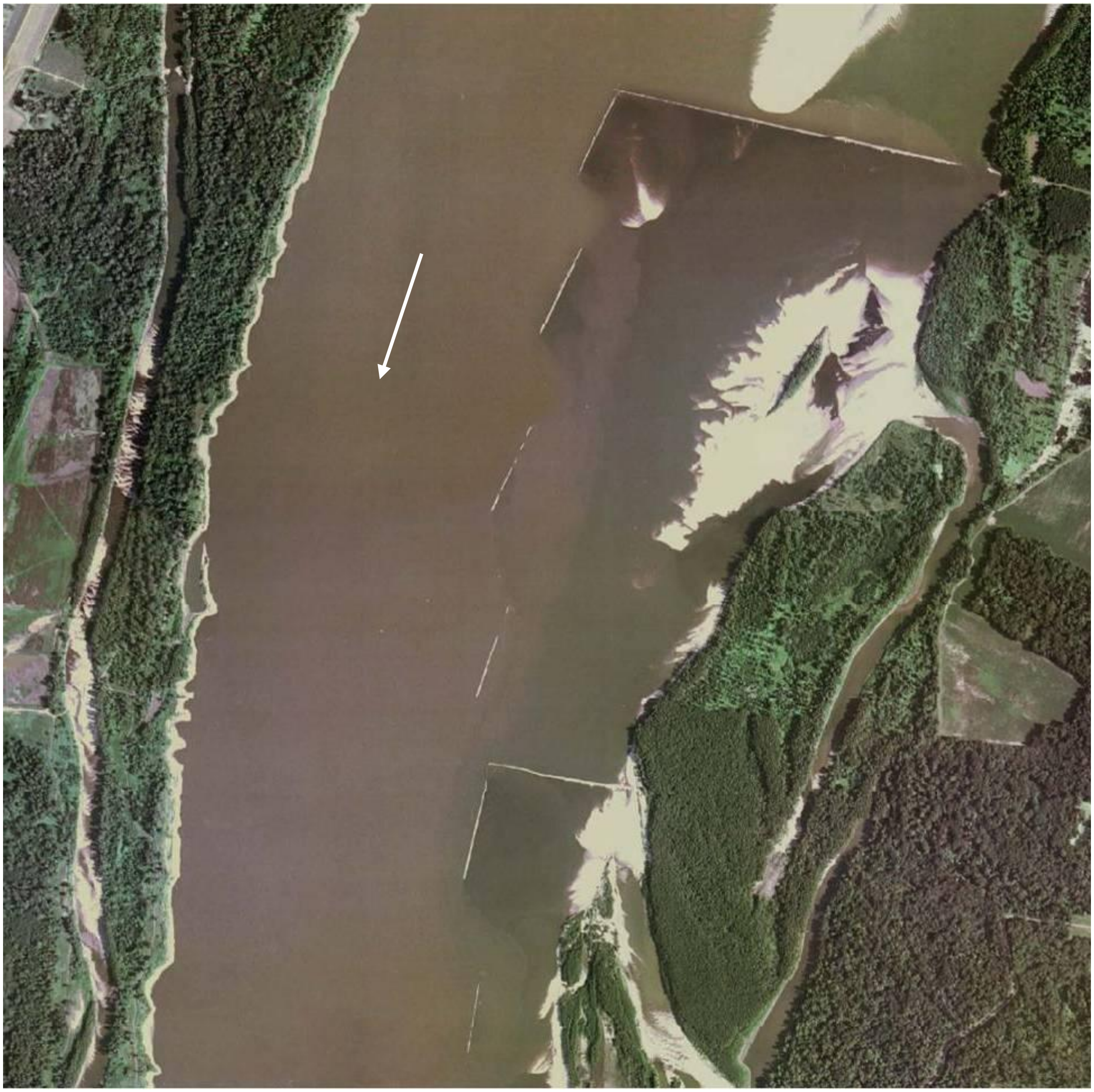

FIGURE 5.11 Vane dikes between L-head dikes on Mississippi River near Providence, LA

5.2.3 L-HEAD DIKES

L-head dikes are dikes with a section extending downstream from the channel ends generally about parallel to the channel line. The addition of the L-head section can be used to increase the spacing between dikes, to reduce scour on the stream end of the dike, or to extend the effects of the dike system farther downstream. The L-head portion of the dike takes the energy at the stream end of the dike and spreads it over a larger area, L-heads tend to block the movement of sediment behind the dike by reducing the formation of eddies (recirculation) in the lee/downstream of the dike. When the L-head crest is lower in elevation than the dike crest, surface currents coming over the top of the L-head can cause scour on the landward side. L-head dikes have also been used to reduce shoaling in harbor entrances or to maintain an opening in the downstream end of a bypass channel. Figure 5.12 is an example of use of L-head dikes to reduce the effects of a major bank line discontinuity. In such an application a longitudinal dike would have been effective also; however, the use of L-head dikes was probably as effective, but at a much reduced cost due to significantly lower quantity of stone required to construct the L-head dikes verses a longitudinal dike.

FIGURE 5.12 L-head dike on the Mississippi River

5.2.4 CLOSURE DIKES

River reaches that include islands and divided flow tend to have limited depths in part due to the loss of energy through the secondary channel. In the past, such cases were modified by reducing or eliminating the low and medium flows from all but the main channel being developed for navigation. This was accomplished by diverting sediment into the side channels or constructing closure structures across the side channels. Sediment could be diverted into the side channel using spur dikes, vane dikes, or a combination of both. Within the secondary channel the closure dikes will further reduce the velocities in the channel and enhance the depositional tendencies in that channel. When the length of the side channel is short relative to that of the main channel, as is the case in a bendway, closure dikes across the secondary channel tend to be difficult to maintain because of the high head differential that develops across the dike and the subsequent scour downstream of the dike. In such cases, closure structures in the secondary channel should have at least two dikes. With the dikes constructed at successively lower elevation moving downstream the total drop in the secondary channel will be divided between structures, which will reduce the amount of scour that would tend to endanger a single structure (Figure 5.13).

FIGURE 5.13 Looking downstream at closure dike on secondary channel on Mississippi River. Note widened bankline just below the dike caused by the plunge pool when the dike is overtopped.

5.2.5 KICKER DIKES

In some instances kicker dikes are provided on the downstream end of revetments to constrict the downstream crossing which slightly increases velocities to eliminate sediment deposition and associated maintenance dredging in the crossing. These kicker dikes provide greater contraction of the channel in channel crossings to help maintain the navigation channel alignment. For example, on the Red River in Louisiana, which has a maintained navigation channel width of 600 feet, kicker dikes were used to constrict the crossing, especially in the upper reaches of the pools to a width of 450 feet (see Figure 5.14).

FIGURE 5.14 Crain Revetment Kicker Dike, Red River. Louisiana

5.2.6 SILLS IN UPSTREAM LOCK APPROACHES

Sills, also referred to as submerged dikes or groins, are constructed in the approaches immediately upstream of the lock walls to reduce velocities in the approach. When a towboat moving along a bank leaves the deeper water typically in the main river channel to a shallower portion immediately upstream of locks the tow blocks a significant portion of the flow causing a higher water level to develop between the bank and tow. Initially this situation may cause the head of the tow and eventually the rest of the tow to move riverward, which creates a difficult navigation condition entering or leaving a lock. By installing sills in the channel upstream of the lock the flow velocities in the area are reduced and the higher velocities are shifted riverward, away from the navigation channel, toward the main river channel.

Sills are constructed similar to stone, spur dikes with some slight modifications since the sills are submerged and towboats navigate directly over the structures. Franco (1976) stated that the "… elevation and spacing of the groins (sills) would depend on channel depths and current direction and velocities." Sills are typically constructed 20 feet below the normal upper pool elevation and spaced one to one and one-half times the length of the sill immediately upstream. Sills extend from the bank at least as far as a line formed by extending the land-side face of the upper guard wall. Franco also recommended not introducing fill between the sills due to reduced effectiveness of the sills by reducing channel roughness and velocity reduction capability. Figure 5.15 shows model study results of sills in a lock approach being navigated by a remote controlled towboat.

Sills have also been used in open river situations, such as on the Missouri River, to reduce velocities in the navigation portion of the channel and improve navigation conditions in local areas. As is the case described above in upper lock approaches, such sills do not significantly impact flow conveyance and only have local effects.

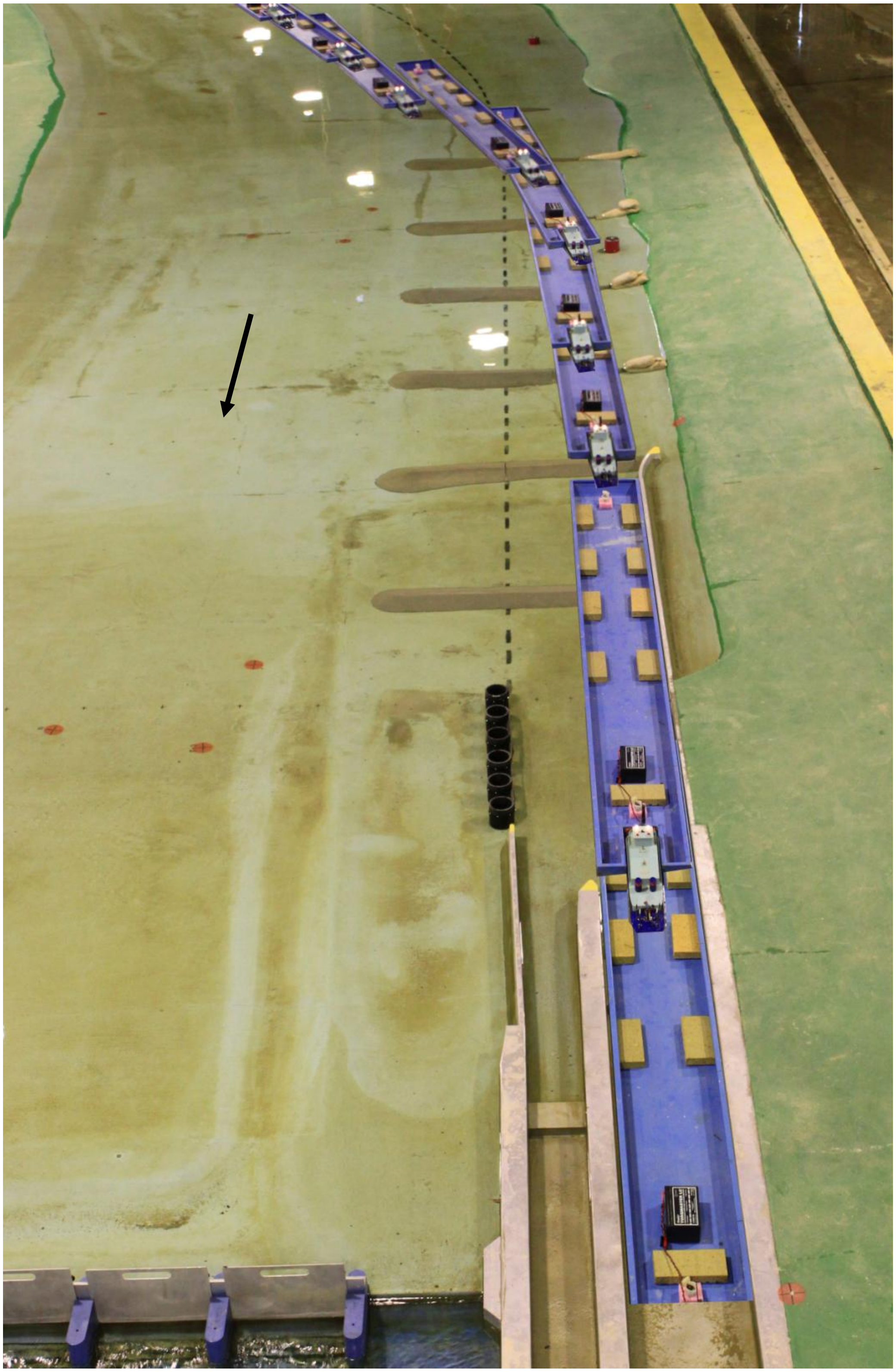

Figure 5.15 Upbound remote controlled tow passing over sills on model study of Montgomery Locks and Dam, Ohio River

5.2.7 CROSS-SECTIONS OF VARIOUS STURCTURES VERSES REFERENCE ELEVATIONS

To better understand the relationship between various types of river training structures, Figure 5.16 is provided below. As can be seen in the figure, the top elevation of a hard point (see Section 7.4) at the bank line would be constructed at approximately the Top of Bank elevation. Spur dikes (see Section 5.1.2) and vane dikes (see Section 5.2.2) are normally constructed at about midbank height or at about elevation +15-ft on rivers such as the Mississippi River. Finally, bendway weirs (see Section 7.1) and sills (see Section 5.2.6) are constructed to elevations low enough so as not to interfere with navigation even at the lowest stages encountered in the reach. Therefore, the bendway weirs and sills are constructed at an elevation such that the loaded vessel draft and an acceptable amount of clearance is used in the design. On the Mississippi River and other major rivers the bendway weirs and sills are constructed 15 to 20 ft below the LWRP in open river systems or normal pool elevation in lock and dam systems.

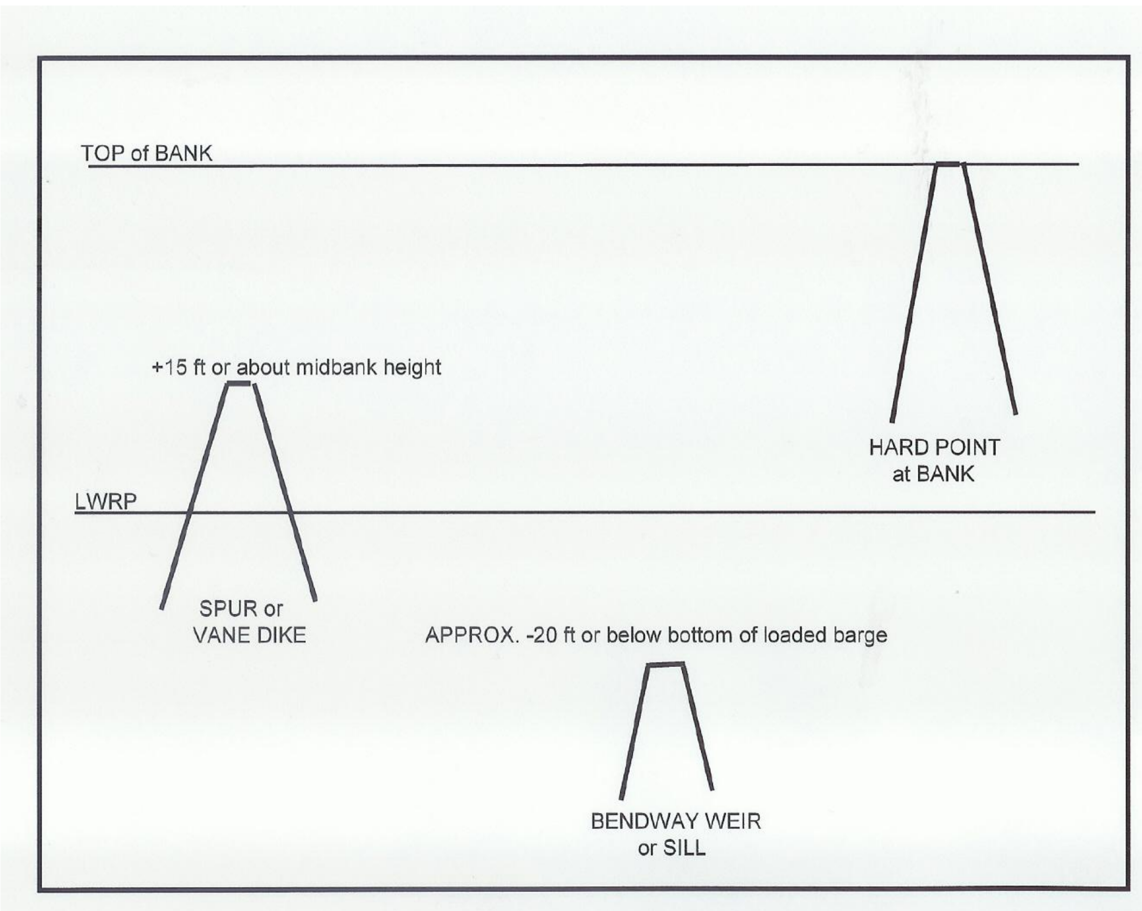

Figure 5.16 Cross-sections of various types of training structures relative to the Top of Bank and Low Water Reference Plane (LWRP) elevations

5.3 TIMBER PILE DIKES

In the United States, the vast majority of the first river training dikes were constructed using timber piling with stringers or spreaders between individual piles or pile clumps to produce an integrated and stable structure. Pile dikes and dike systems were instrumental on streams carrying large amounts of fine-grained sediments such as the Missouri River to accumulate sediment within the dike fields. Once the bars between the pile dikes began to fill the dikes were filled with stone to increase the life and usefulness of the system. Pile dikes were susceptible to damage from ice, fire, impacts from vessels and large debris, and the inevitable process of deterioration and rot. Therefore, river engineers leaned more and more toward using stone to construct dikes and the use of pile dikes and the similar channel training pile revetments (discussed in Chapter 7) decreased over time. However, on some systems like the Columbia River in the Northwest where timber was always readily available, pile dikes continued to be used. There the pile dikes are a hybrid of a stone dike (enrockment) emplaced along the river bed to form the lower cross-section of the dike and timber piles placed through the enrockment to form the upper cross-section of the dike.

The development of navigation on the Red River in Louisiana is one recent project of the U.S. Army Corps of Engineers which made wide use of timber pile dikes and revetments for the initial river training structures. This section will present some of the details used by the Corps of Engineers Vicksburg District in the Red River development as well as present a summary of the application of timber pile dikes on the Missouri River.

On the Red River pile dikes with stone fill (also referred to as stone-filled pile dikes) were used to control the river, train it to the rectified alignment, promote the development of new banks, close off secondary channels, and serve as baffles landward of pile or stone-fill revetment. The type used in a specific location, as with revetments, depends on the existing channel conditions and economy of construction.

The theory behind the Red River stone-filled pile dikes was to have the pile portion of the structure designed to trap fill and minimize subsequent stone-fill quantities, so the piling is generally untreated. In the initial phase, the pile structure is constructed and a low, stone fill is placed in the dike line to protect the base of the piling from scour, add to the structural support of the piling, and aid in the creation and protection of sediment deposits. After accretion has formed and prior to deterioration of the untreated piling, which usually rots out in 5 to 10 years, additional stone (cap-out) is placed landward of the piling. This may be done in more than one phase, depending on the rate of accretion and condition of the piling.

The major components of the Red River stone-filled pile dikes are:

- Piles - Southern yellow pine, Spruce, or Douglas fir. In most instances untreated piling were specified since most structures were capped-out with stone after accretion had filled along or behind the structures.
- Stone-fill – quarry-run stone which was well graded and had a maximum size limited to 1,000 pounds. The stone was sufficiently large to withstand the high velocities to which the dikes would be subjected. The stone needed to be well graded to form a dense stable mass so as to minimize undermining damage resulting from seepage. Also, this size stone was required to prevent damage to the piling and to form a dense mass in and around the pile clusters.
- Strand wire – used for making all ties was copper-coated steel strand or equally corrosion-resistant material.
- Staples – used were copper-coated steel cut from 1/8-inch stock with two legs of equal length, each leg being 3.75-inches long and a distance of 0.75-inches apart.

- Bank heads and stone roots – were used to provide connections with the existing bank line. A bank head was used where a steep bank was encountered and a stone root was used where the bank was flat or where the dike was constructed channelward of a relatively flat, sloping sand bar.

Stone-filled pile dikes were designed and constructed based on the specific conditions they were subjected to and the purpose of the particular dikes. The crown width and downstream slope depended on the degree of attack and pattern of the flow, and on the head differential across the structure. Chute and abandoned bendway closure dikes in particular were designed with a broad base to eliminate undermining by providing a sufficiently long seepage path to impede or prevent flow through and beneath the structure. A stone-fill dike was frequently built with the crown at a low elevation initially, either because of hydraulic considerations, stability of the structure, or economy, and then raised to ultimate height at a later time when the original conditions had altered.

The top of the piling for timber pile dikes (and revetment) on the Red River was set at +17 feet ALWP or 5 feet above the minimum pool, whichever was higher. The stone-filled pile dikes consisted of a 2-or 3-row, 3-pile clump or 2-row single pile structures, depending on the severity of attack, with stone fill. This type of structure was the most economical structure to construct to accomplish the major realignments necessary on the Red River. Figure 5.17 presents design details for Red River stone-filled pile dikes (USACE, 1972).

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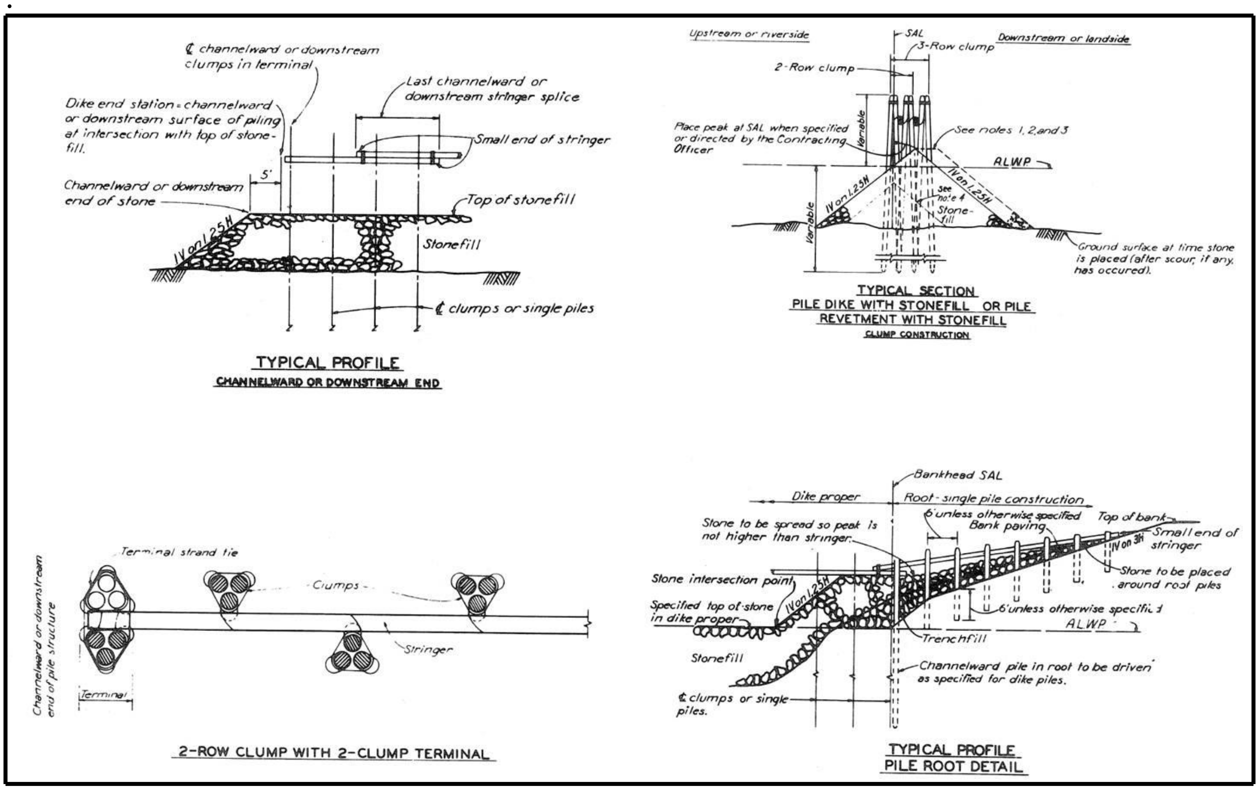


FIGURE 5.17 Red River Pile Dike Details

Construction of the navigation channel on the Missouri River was initiated in 1928, and the navigation channel was originally intended to be approximately 760 miles long. The channel was to extend from Sioux City, Iowa to the confluence of the Missouri River with the Mississippi River just north of St. Louis, Missouri (USACE, 1947). Due to cutoffs and channel realignments the navigation channel from Sioux City to the Missouri-Mississippi confluence ended with the present length of about 734 miles. The Missouri River navigation channel was designed using timber pile dikes and revetments since it was felt that these types of structures provided the most economical and practical method to handle the large sediment load on the river. The Missouri River navigation channel is 300-ft wide with a minimum depth of 9 ft.

The timber pile dikes were used to direct the river flow as well as control the river while the timber pile revetments stabilized the bank lines were necessary. Cutoffs and channel realignments were also included in the Missouri River master plan In the 1947 U.S. Army Corps of Engineer report on the Missouri River it was stated that “The navigation structures on the Missouri River below Sioux City are stabilizing the river course, preventing shifting of the channel and erosion of farm lands and farm levees, and reducing the damage to highways, railroads, and public works. The deposition of silt behind the dikes is also permitting the reclamation of formerly useless channel areas for agricultural use and creating thousand of acres of excellent wildlife habitat” (USACE, 1947).

Initially the timber pile dikes constructed on the Missouri River were single pile in line using treated piles. As the project evolved and sediment accumulated within dike fields stone was placed in some of the timber pile dikes to control flow through the dikes and manage the sediment within the dike fields. This also provided a long-term stability to the dikes as the timbers deteriorated. As the navigation channel became established and developed on the Missouri River, the actual approach to dike construction was modified to work with the channel that was forming. The timber pile dike design was changed to a stone-filled timber pile dike design. It was found that using stone-filled timber pile dikes was a cost effective method that provided a structure with a longer life. It was determined that this type of structure provided savings by allowing the stone to be placed at a steeper slope within the timber piles than an all-stone dike where the natural angle of repose of the stone would determine the side slopes. The Corps’ Committee on Channel Stabilization presented the modified construction sequence used on the Missouri River stone-filled timber pile dikes (USACE, 1969a). The construction procedure was:

- Untreated piles for the dike are driven,
- Stone is placed around the piling to the CRP or above,
- Later, following deposition against the dike, stone is added in the dike on the top and side where fill occurred,
- Therefore, the additional stone does not extend to the original pile dike base, but only to the bed elevation of the deposition

Thus the cost savings realized by adopting the modified design of pile dikes on the Missouri River were realized by using untreated piles on the stone-filled timber pile dikes and raising the dike to final grade on sediments deposited within the dike field with reduced volumes of required stone. Figure 5.18 shows the Missouri River stone-fill timber pile dike details as described above.

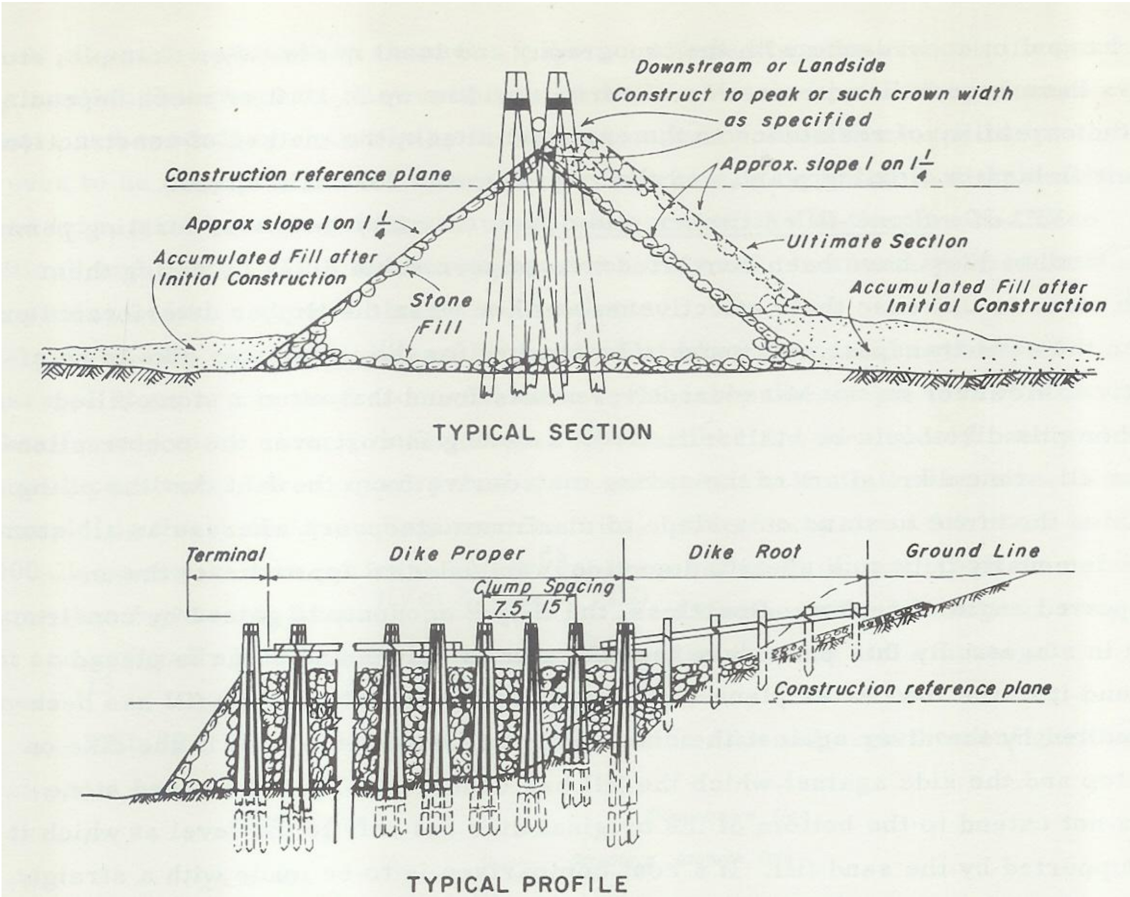


FIGURE 5.18 Missouri River Stone-Filled Timber Pile Dike Details

The following series of photographs show the overall effectiveness of the pile dike application at the Rock Bluff Bend, Nebraska on the Missouri River. It should be noted that the orientation of the photographs (Figures 5.19 through 5.24) is looking upstream. Figure 5.19 shows the reach in September 1934 prior to the construction of any dikes. Figure 5.20 shows the Rock Bluff Bend in March 1935 after construction of the timber pile dike field in the reach. Figure 5.21 shows the reach in October 1939 after the timber pile dikes were in place a little over 4 years. Note the significant shoaling that had occurred within the dike field extending from the bankline over approximately one-half of the length of the dikes. Figure 5.22 shows the Rock Bluff Bend reach in September 1942 with a longitudinal, stone-filled timber pile dike (or revetment) connected to the streamend of the timber pile dikes on the outside of the bend and the addition of timber pile dikes on the point bar on the other side of the river. Figure 5.23 is a photo of the Rock Bluff Bend reach in 1956 showing the dike field on the outside of the bendway converted to farm land and the longitudinal, stone-filled timber pile dike essentially revetment protecting this land. Also, the timber pile dikes on the opposite point bar have fully stabilized the navigation channel alignment through the reach. Figure 5.24 shows the Rock Bluff Reach in March 1983 indicating the overall stability of the reach using timber pile and stone-filled timber pile dikes.


FIGURE 5.19 Uncontrolled Section of Rock Bluff Bend, Missouri River, September 1934


FIGURE 5.20 Rock Bluff Bend, Missouri River with Timber Pile Dikes, March 1935

FIGURE 5.21 Rock Bluff Bend, Missouri River Showing Deposition in Timber Pile Dike Field, October 1939

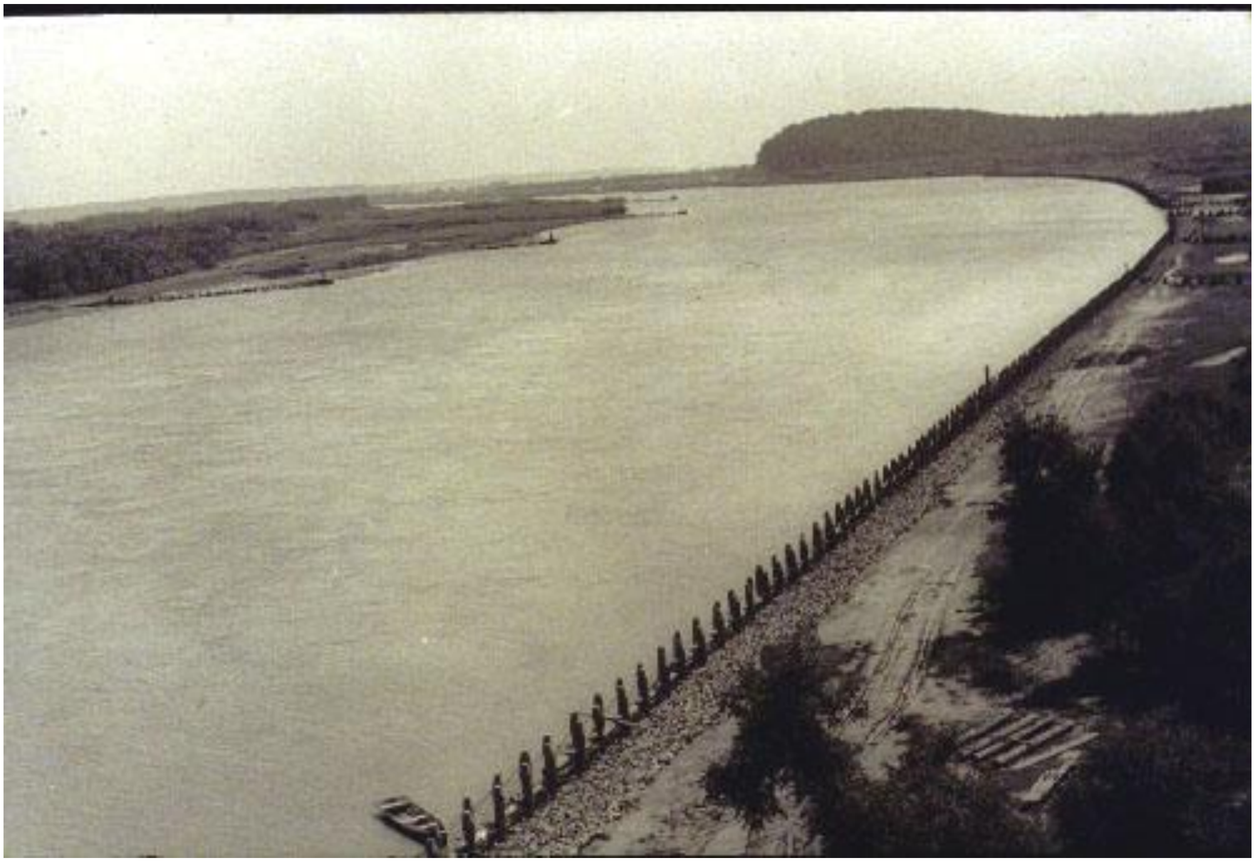


FIGURE 5.22 Rock Bluff Bend, Missouri River Longitudinal Stone-Filled Timber Pile Dike Added and Timber Pile Dikes on Opposite Point Bar, September 1942


FIGURE 5.23 Rock Bluff Bend, Missouri River, 1956


FIGURE 5.24 Conditions at Rock Bluff Bend, Missouri River, March 1983

5.4 INVENTORY OF TRAINING DIKES ON U.S. RIVERS

An inventory of river training dikes on the U.S. inland waterway system was made for a U.S. Army Corps of Engineers' research program entitled "Repair, Evaluation, Maintenance, and Rehabilitation" (REMR). This inventory was completed in 1988, and although it may seem somewhat outdated, it provided a good basis for updating the comprehensive inventory of river training dikes. This inventory of shallow-draft training structures was generated by a literature search of Project Maps of U.S. Army Corps of Engineer Districts within the continental United States. This information provided the foundation for an extensive data base of these types of structures and identified 10,652 training structures located in shallow-draft, nontidal-influenced riverine areas (see Derrick, 1989). The research indicated the following summary delineated by various structure categories:

- 6,132 stone dikes
- 2,979 stone-filled pile dikes
- 1,503 pile dikes
- 35 closure structures
- 2 sills
- 1 control weir

Updating the information in the 1989 REMR report referenced above results in the following number of dikes and bendway weirs by river system and location within specific U.S. Army Corps of Engineers Districts:

Corps of Engineer District		No. of Dikes/Bendway Weirs
Mississippi River and Tributaries System		
Kansas City		3,560
Little Rock		1,018
Memphis		273
Omaha		2,833
Rock Island		1,348
St. Louis		1,825
Pooled Portion Dikes	440	
Pooled Portion Bendway Weirs	4	
Open River Portion Dikes	1,205	
Open River Portion Bendway Weirs	176	
St. Paul		1,329
Tulsa		114
Vicksburg		398[1]
Ohio River and Tributaries System		
Louisville		33[2]
Huntington		15[3]
Columbia River and Tributaries System		
Portland		3[4]
Other River Systems		
Savannah		253
Mobile		153
TOTAL		13,155

[1]Includes 35 closure structures, 1 control weir, and 2 sills.
[2]Includes 7 closure structures
[3]Includes 6 closure structures
[4]The lower Columbia River has an additional 258 timber pile dikes (not included in the above total) which have some tidal influence; however, the dominant flow is fresh water from the upper Columbia River.

CHAPTER 6

REVETMENTS

6.1 REVETMENT STRUCTURES

Revetments are structures to resist the erosive forces of moving water and are normally used on the outside of bends on meandering navigation channels. Figure 6.1 shows a typical revetment application consisting of Articulated Concrete Mattress (ACM) and upper bank stone paving.

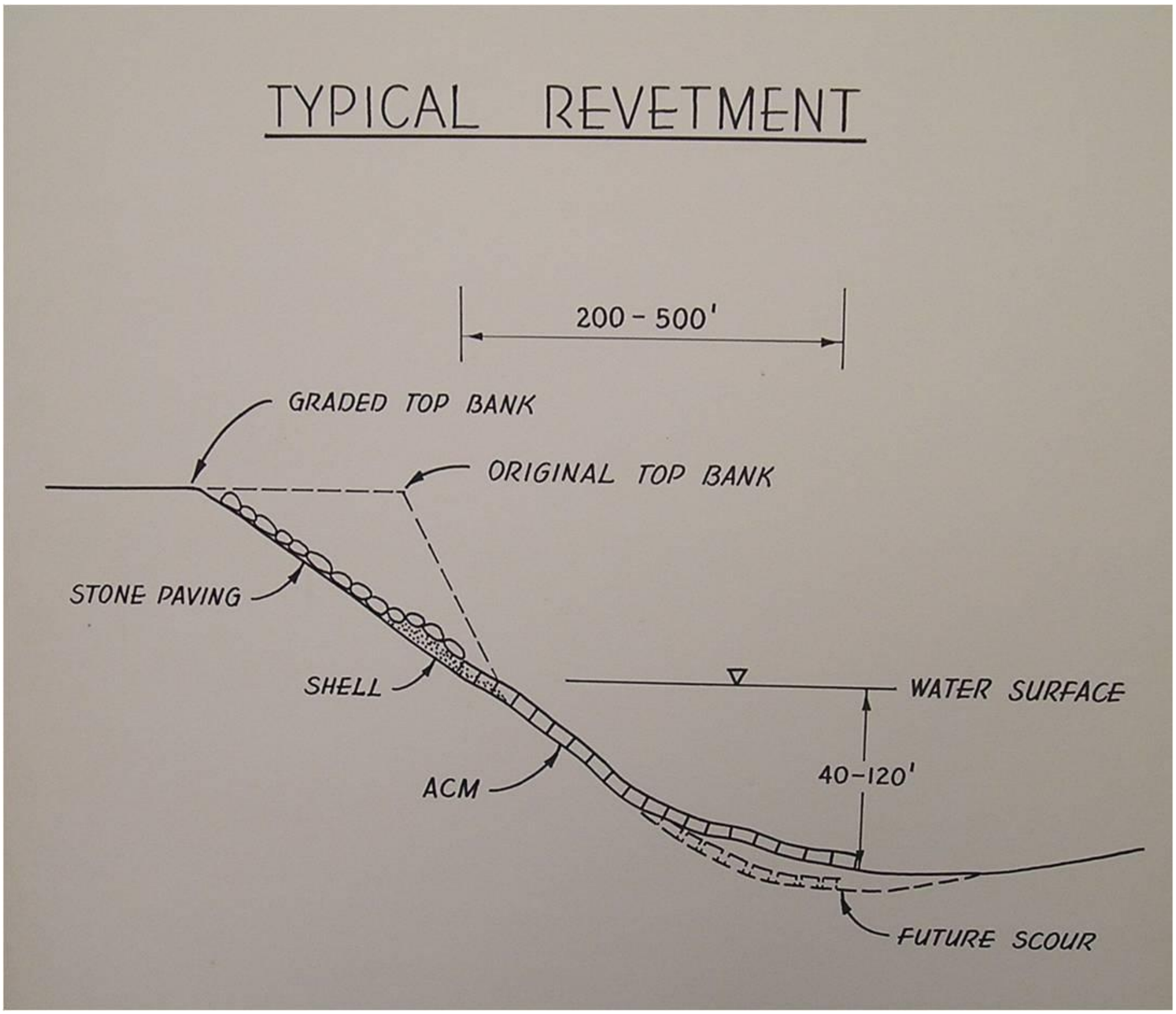


FIGURE 6.1 Sketch of typical river bend revetment

6.2 STONE REVETMENTS

Stone, or riprap, is the common material used to build revetments. "Rock" is considered the proper name for in-ground material. Once the rock is excavated and reduced to smaller fragments, it is considered "stone".

The benefits of stone as a revetment building material are:

- Generally local availability
- Can be moved by truck or barge
- Failure is usually localized and catastrophic failure of the whole structure does not occur

6.2.1 TRADITIONAL STONE REVETMENT

The majority of stone revetments consist of a layer of non-uniform size stone laid on a sloping river bank. Geotechnical analysis is typically required to determine the appropriate stable bank slope. The following criteria are extracted from USACE 1994:

Riprap should be blocky in shape rather than elongated, as more nearly cubical stones "nest" together best and are more resistant to movement. The stone should have sharp, angular, clean edges at the intersections of relatively flat faces.

The gradation of stones in riprap revetment affects the riprap's resistance to erosion. Stone should be reasonably well graded throughout the in-place layer thickness. Specifications should provide for two limiting gradation curves, and any stone gradation as determined from quarry process, stockpile, and in-place field test samples that lies within these limits should be acceptable. Riprap sizes and weights are frequently used such as D_{30} (min), D_{100} (max), W_{30} (min), etc. The D or W refers to size or weight, respectively. The number is the percent finer by weight. The (max) or (min) refers to the upper or lower limit gradation curves, respectively. The gradation limits should not be so restrictive that production costs would be excessive. The choice of limits also depends on the underlying bank soils and filter requirements if a graded stone filter is used.

Rather than a relatively expensive graded riprap, a greater thickness of a quarry-run stone may be considered. Some designers consider the quarry-run stone to have another advantage: its gravel and sand-size components serve as a filter. The gravel and sand sizes should be less by volume than the voids among the larger stone. This concept has resulted in considerable cost savings on large projects such as the Arkansas and Red River Navigation Projects. Not all quarry-run stone can be used as riprap; stone that is gap graded or has a large range in maximum to minimum size is probably unsuitable.

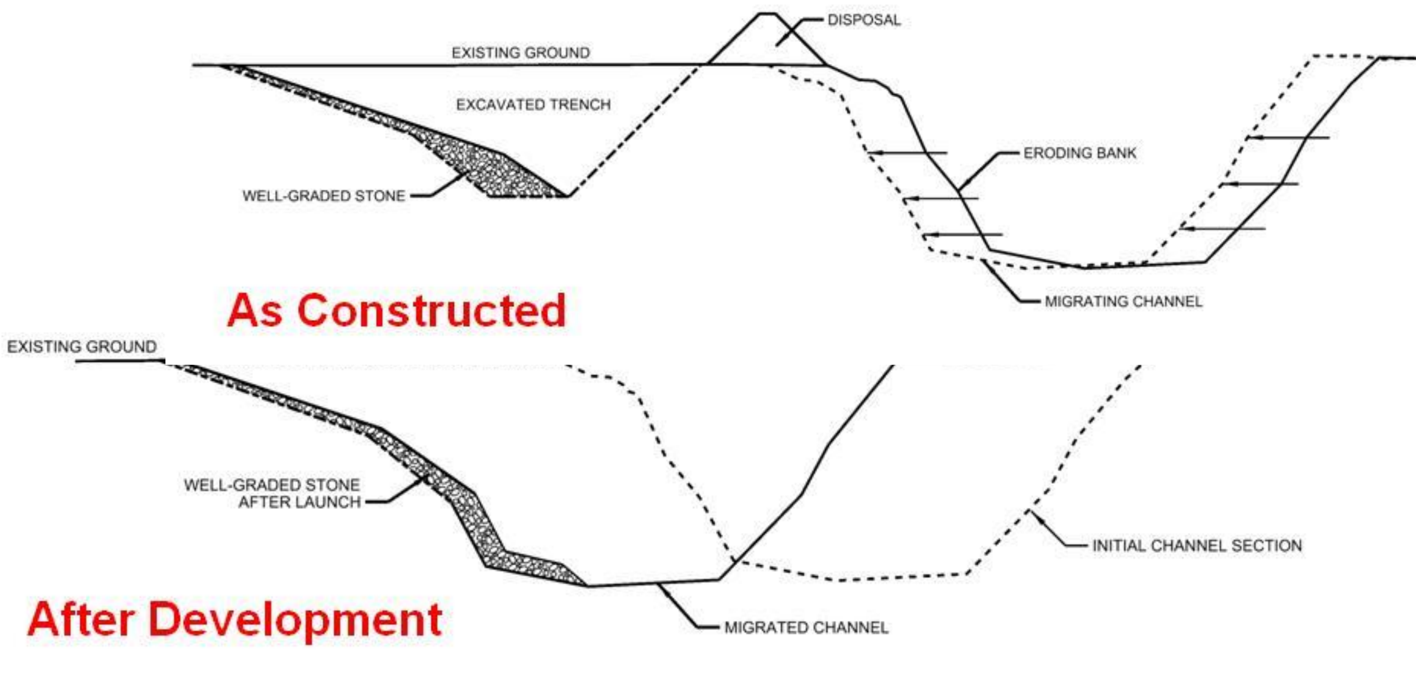
The stability of riprap slope protection is affected by the steepness of channel side slopes. Side slopes should ordinarily not be steeper than 1V on 1.5H, except in special cases where it may be economical to use larger hand-placed stone keyed well into the bank. Embankment stability analysis should properly address soils characteristics, groundwater and river conditions, and probably failure mechanisms. The size of stone required to resist the erosive forces of channel flow increases when the side slope angle approaches the angle of repose of a riprap slope protection. Rapid water-level recession and piping-initiated failures are other factors capable of affecting channel side slope inclination and needing consideration in design.

Revetment techniques vary depending on the region and the local materials available. Revetments within St. Louis District USACE consist of placing a minimum of 30-inch rock blanket of "A" stone (a well-graded stone with maximum size of 5,000 pounds) on the existing bank grade. If the existing bank grade is less than a 1V on 1.5H then stone will be placed until a 1V on 1.5H is met. St. Louis District typically does not grade the banks prior to revetment placement or use any filter fabric or geotextile.

The riprap bank protection on the 31.7 mile long Divide Cut on the Tennessee-Tombigbee Waterway is 18 inches thick and is placed on filter fabric. The well graded stone has a 356 lb. maximum size and a 12 lb. minimum size. Details of this revetment are provided in Chapter 8, case studies.

6.2.2 TRENCHFILL REVETMENT

Trenchfill revetment is used either along existing banks or for construction in an excavated trench where the rectified alignment is landward of the natural existing bank. This structure is most advantageously used on banks of relatively non-cohesive material where scour adjacent to the trench and subsequent launching of the stone will be more uniform. Trenchfill revetments have been used very successfully on the Arkansas River, Mississippi River, and Red River in Louisiana, and where the details presented herein may not be universally applicable they do provide a basis for other systems. Typical details are shown in Figure 6.2.



A trenchfill structure is designed so that the stone in the toe trench is launched to pave the underwater bank as the streambed adjacent to the toe is lowered by the scouring action of the river. Sufficient stone to pave the underwater slope to a depth of about 50 feet below the ALWP can be provided in the toe trench; however, it is usually more economical to shape the alignment to limit the anticipated depths of scour to 20 or 25 feet below the ALWP. The bottom of the trenchfill is generally placed as low as practicable for optimum protection, usually to a depth of 4 to 8 feet below the ALWP. Dewatering equipment will be used as necessary to excavate the trench.

Above the trenchfill, the bank will be graded to a 1V on 3H slope and paved to about 17 feet above the ALWP or 5 feet above pool level, whichever is higher. The graded bank above the bank paving, the berm, the riverward slope, and an adjacent 5-foot strip on the landward slope of the spoil bank will be seeded for erosion control. On the Red River in Louisiana, the material excavated from the trench was disposed between the trench and top bank of the river so that it would be eroded and transported downstream as the top bank continued to migrate toward the trench (USACE 1972).

6.3 OTHER REVETMENT TYPES

Beside the more typical types of revetments discussed above, other types of revetments such as board mattress, timber piles, and articulated concrete mattress are also placed on rivers. Early efforts of bank stabilization used willow mattresses (similar to board mattresses) extensively, and after being in place for over 100 years those structures are still functioning and providing bank protection. Board mattress, timber piles, and articulated concrete mattress are discussed in the following sections.

6.3.1 BOARD MATTRESS REVETMENT

Standard board mattress revetment is placed along existing banks and is particularly effective on banks of highly cohesive material where bank recession would otherwise occur by the soil below the waterline being washed away, undermining the upper bank. This type revetment is effective only where existing bank alignment and channel depths are adequate for navigation. An example of board mattress revetment is shown in Figure 6.3.

FIGURE 6.3 Section of board mattress revetment under construction

After fabrication, the board mattress is placed on the underwater bank so as to extend from the water surface at the time of construction to riverward of the thalweg. The board mattress is sufficiently flexible to adjust to irregularities in the existing underwater slopes or to changes which would occur in the underwater slope subsequent to construction. The mattress is ballasted with 2 tons of stone per square (100 square feet), with additional stone placed in cribs constructed around the perimeter and through the longitudinal axis of the mattress. In locations where unusually severe attack may be expected, as in sharp bends, the stone may be increased to 5 tons per square.

Above the mattress, the upper bank is graded to a 1V on 3H slope and is paved with a layer of graded stone to about 17 feet above the ALWP or 5 feet above pool level, whichever is higher. The graded bank and overbank spoil will be seeded. (USACE, 1972)

6.3.2 TIMBER PILE REVETMENT

On the Red River in Louisiana stone-filled revetments were very successful in developing the system in certain critical conditions. On low banks, low bars, and where the rectified channel control lines crossed the existing channel, either timber pile revetment with stone-fill or stone-fill revetment was used, again depending on conditions and economy of construction. The latter two types of revetment were designed to induce deposition of sediment, to create new bank lines, or build up existing banks and to encourage willow growth on the accretions. If the rectified channel control line was a considerable distance from the existing bank, baffle dikes are used landward of the revetment to provide an additional screening effect to induce deposition.

The general design is virtually identical to the stone-filled pile dikes described in Section 5.3. Figure 6.4 is a view of stone-filled pile revetment on the Red River.

FIGURE 6.4 Photographs of Red River in Louisiana pile revetments – Immediately after construction and after sediment deposition had occurred

6.3.3 ARTICULATED CONCRETE MAT (ACM) REVETMENT

ACM is a specialized type of revetment used on deep channels (50 feet deep or deeper) to stabilize the channel alignment. Their use is limited to large rivers, like the lower Mississippi River. ACM is used on eroding bendways where water depths are typically 40 to 120 feet deep, so the mat length under water can be between 200 and 500 feet. ACM revetment schematic and sinking photographs are shown on Figure 6.5.

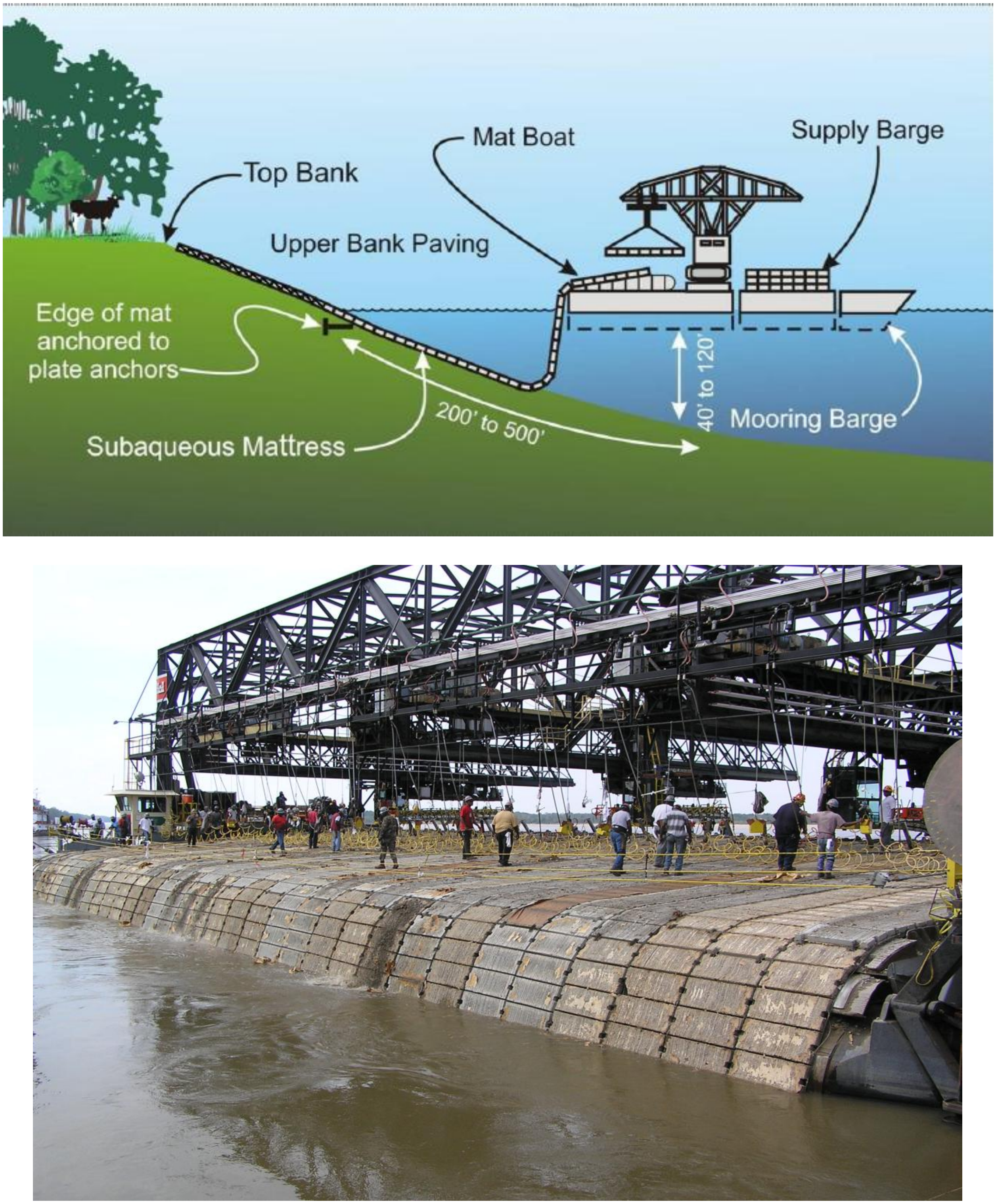


FIGURE 6.5 ACM Revetment Schematic and Sinking Photograph

The use of ACM is restricted to large rivers due to the size of the floating plant required to place the mat. Therefore, in the Vicksburg District, ACM has only been used on the

Mississippi River, upper Atchafalaya River, and on the lower Red River below the mouth of the Black River. The mat placement process includes several phases. The first phase involves the clearing of the bank. Many of the banks on the lower Mississippi River are covered with vegetation that must be removed. Caving banks are usually very steep. Therefore, to insure the stability of the revetment, the bank is graded to predetermined design slope using borings taken from near top bank. The banks are graded by a large floating diesel-electric powered dragline. The clearing and grading of the banks are conducted on a schedule so that work is completed prior to the arrival of the mat-sinking unit. The time between grading and sinking varies, early in the construction season there can be a long delays of from 4 to 6 weeks while near the end of the season the delay may be only a couple of days. Crushed stone is placed on the freshly graded bank to protect it from erosion prior to the placement of the ACM. This minimizes the potential for erosion of the freshly graded bank, reducing the overall cost of the operation and producing a better product. Natural vegetation re-establishes itself quickly along the stabilized top bank following completion of the revetment construction.

The ACM consist of individual units of mat that are pre-cast and stored at strategically located casting fields along the river. Each unit contains 16 separate concrete blocks that are held together by corrosion-resistant wire embedded in the neutral axis of the concrete. Each block is 4 feet long, 18 inches wide, and 3 inches thick, resulting in each unit being 25 feet long by 4 feet wide. The surface of the mat is roughened during the casting process by dragging a large wire brush type tool over the uncured concrete after it is placed into forms. Use of the roughened mat improves the aquatic habitat provided by the river by increasing the surface area for growth of macroinvertebrates which retain and recycle organic materials that would otherwise be lost from the system as well as providing a food source for fish and birds with recreational, ecological, and economical value.

Once the revetment season begins, the ACM units are loaded onto barges and transported to the construction site. At the site, the units are loaded on a specially designed mat-sinking barge. The units are wired together into a mattress that is 140 feet wide and anchored to the bank with cables. As the mat is assembled, the sinking barge (aka the “mat boat”) moves out into the river along the mooring barge. As the sinking barge moves, the ACM is launched off the barge and covers the river banks and bed. Placement of the mat continues to beyond the deepest part of the channel. The mat sinking plant is then moved upstream to lay the next section of mattress. This process continues with each succeeding mattress overlapping the previous mattress in a manner similar to shingles on a roof until the desired length of bank has been revetted. Once the ACM is in place, placing stone riprap on the graded upper bank completes the revetment process.

ACM is placed during the low water season. On the lower Mississippi River, the revetment season typically extends from early August until all scheduled revetment placement is complete, usually in November. The location and amount of revetment placed each year is based on a prioritization of needed stabilization work and available funding (Pinkard, 2001b). Through 2010, over 296 miles of revetment have been placed on the Mississippi River within the Vicksburg District with only 10 miles of revetment remaining. Therefore, the revetment construction program is approximately 97 % complete. However, maintenance ACM is placed on a regular basis, as needed, over previously placed sites.

On the Lower Mississippi River it is estimated that 3 to 5% of the total surface area of aquatic habitat is revetted, and it is estimated that when the Mississippi River and Tributaries Project is completed almost 50% of the Lower Mississippi River bankline will be revetted (USACE, 1988a). In the Environmental Program study it was determined that when ACM is placed on a natural bankline the invertebrate community shifts. Such areas originally colonized by borrowing mayflies are then colonized by macroinvertebrates. Part of this change was a result of an effort initiated in the 1970's where the surfaces of the ACM blocks were roughened while the blocks were in the form. The Environmental Program study stated that relative to these ACM modifications – "In an experiment to determine the affect of ACM surface modifications upon benthic macroinvertebrates, ACM experimental blocks having numerous parallel grooves harbored denser populations than those with holes or commercial 'Fish-hab' or control ACM blocks."

CHAPTER 7

OTHER TYPES OF TRAINING STRUCTURES

In recent years, a number of new and innovative river training structures have been developed. These include Bendway Weirs, Blunt Nosed Chevron, Bullnose Structures, and Hard Point Structure. A brief description of these structures is provided in this chapter.

7.1 BENDWAY WEIRS

One of the newest developments in river training structures involves the concept of bendway weirs to widen the channel in bendways. Typically the natural riverine processes will create a point bar on the inside of the bend. In certain instances the point bar will encroach into the navigation channel, requiring maintenance dredging to widen the channel and restore the design channel dimensions. Bendway weirs are submerged sills constructed in the navigation channel (within the channel control lines) angled upstream. The structures direct flow toward the point bar, eroding it, to use the river energy to widen the navigation channel.

These weirs are designed such that tows may pass directly over these structures at all river stages. These structures are used to direct energy away from the outside bend and are used to maintain uniform channel width and alignment around a bendway. Bendway weirs on the Mississippi River are typically constructed out of “A” stone (well graded 5,000 lb. maximum size), with a 10 foot crown width. These structures typically have a 1V on 1.5H side slope on the upstream end and 1V on 4H side slope on the downstream end. The St. Louis District of the U.S. Army Corps of Engineers constructs weirs downstream to upstream to save stone. Constructing the bendway weir field from the upstream direction creates scour downstream in locations where successive weirs will be constructed. These structures are not typically keyed into the bank because they are usually constructed on the outside of bendways that are typically revetted and protected. It should be noted that not all bendway weirs on the Mississippi River are constructed to these exact specifications. Design is site specific depending on site conditions and some of these parameters may vary slightly as necessary. Bendway weirs are level crested with an elevation of 15 to 20 feet below low water. At this depth, it can be assured that these structures would not interfere with tows drafted at 9 feet during low water conditions. Special care should be taken when designing these structures because tows will be navigating directly over these structures. Engineers need to consider the radius of the bend, the shape of the entrance and exit of the bend, the depths in the bend, the shape of the point bar, velocity distribution at all stages, etc. Physical modeling of the structures is recommended to optimize the design. Figure 7.1 shows an artist’s conception of a bendway weir layout.

The bendway weir concept was initially developed using a physical movable-bed model at the U.S. Army Corps of Engineers Engineer Research and Development Center (ERDC) in Vicksburg, Mississippi. An ERDC report (see Derrick, et al, 1994) provides detailed information on the development of the bendway weir methodology. Since the first bendway weir study was conducted for the St. Louis District, the District was the first organization to install bendway weirs in a navigation channel. The District probably also has installed the most bendway weirs in the United States and has a wealth of information and experience in the application of bendway weirs on the Mississippi River. The District has provided a website (http://www.mvs.usace.army.mil/arec/documents/Files/BendwayWeirDesignManual.pdf) that provides a very comprehensive and detailed presentation of the development and application of bendway weirs. This reference provides an excellent source of design information for river engineers in the use of bendway weirs in a navigation channel.

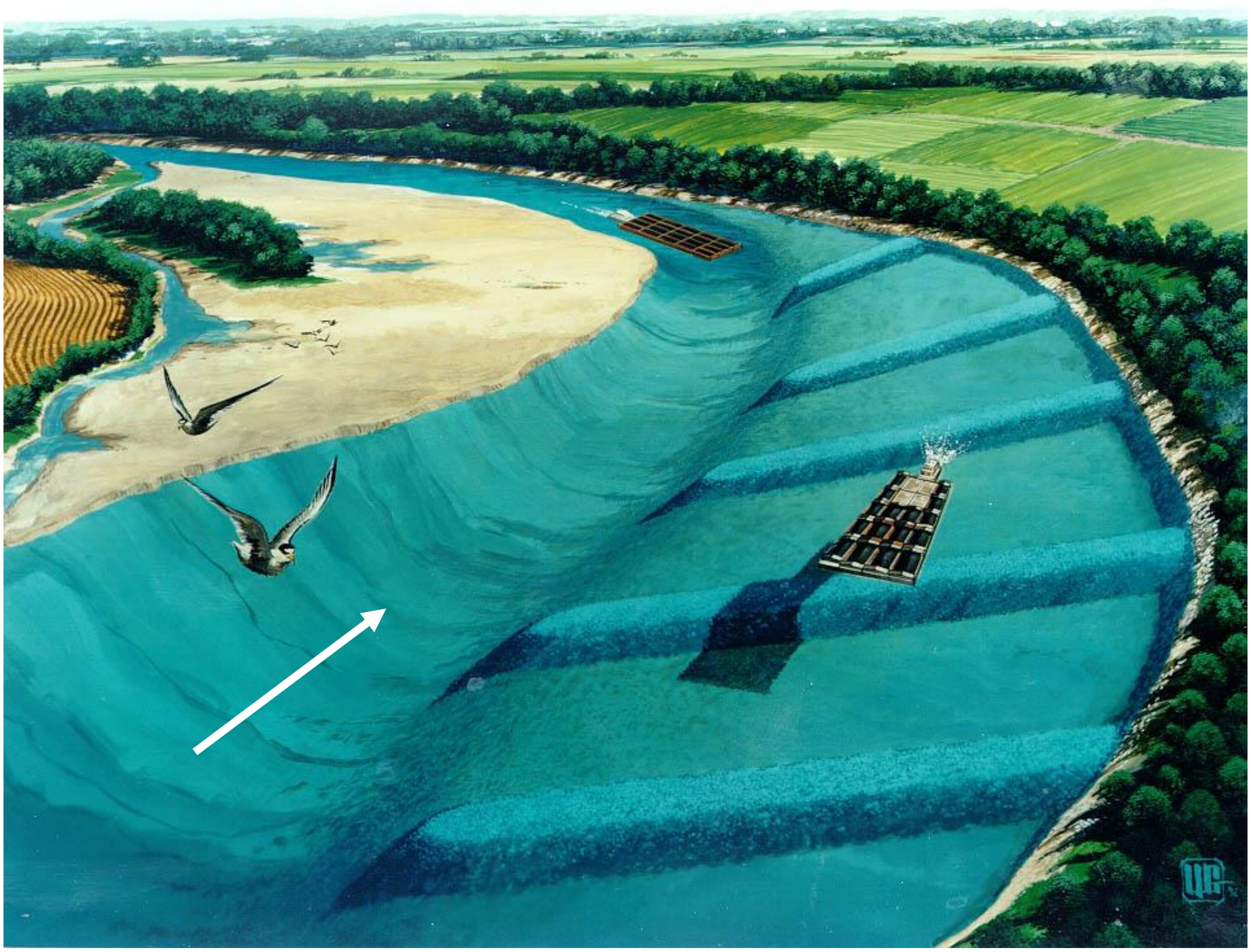


FIGURE 7.1 Artist's conception of a bendway weir field

7.2 BLUNT NOSED CHEVRON

Blunt Nosed Chevrons have an ellipse shaped head and two legs that extend downstream, parallel to the flow and are used to maintain the navigation channel. There are many cases where chevrons have been used for navigation purposes, such as where split flows and channels are necessary in harbor or fleeting areas.

The size of these structures depends on the size of the river and location, the contraction width, and the purpose. Most chevrons on the upper Mississippi River have footprints between 300 feet by 300 feet or 200 feet by 200 feet, which means legs 200 to 300 feet long and width (leg to leg) of 200 to 300 feet. The chevron has a top elevation set to allow overtopping during high flows (floods). The overtopping flows create a scour hole downstream from the structure which provides excellent fish habitat. Figure 7.2 shows a typical chevron plan layout and Figure 7.3 shows this type of structures in place on the Mississippi River. There are many variations of chevrons, typically for site specific reasons and conditions or for additional environmental benefits. Variations include notches at the head of the structure, notches along one or both legs of the structure, differing leg lengths, parallel legs, and flared legs. Some chevrons are lined up parallel with the flow while others are offset to one another.

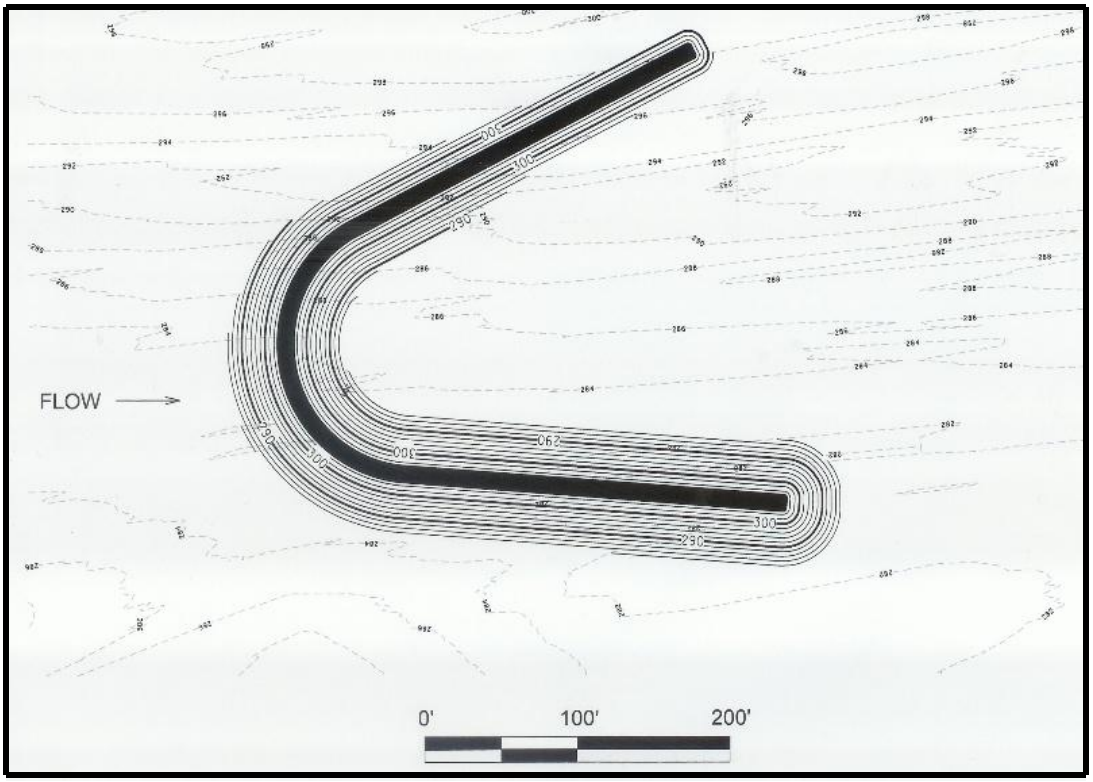


FIGURE 7.2 Chevron Layout

FIGURE 7.3 Chevron Field

7.3 BULLNOSE STRUCTURES

Bullnoses are ellipse shaped structures that provide stability to heads of islands and are designed similar to Off-Bankline Revetments (see Section 10.8). The Bullnose Structure is a row of "A" stone constructed to existing bank elevation and offset from the head of an island. The offset distance is site specific and determined as to not adversely impact the island head when the structure is overtopped. The offset distance varies depending on the size of the structure but is typically no less than 100 feet from the head of the island. Notches are also included in the Bullnose Structures to allow fish access to the slack water areas.

Many times the shape of the bullnose is dependent on the original shape of the island head prior to erosion. This condition is typical when a stable navigation channel existed prior to the island head and the navigation channel deteriorated as the island eroded. The reconstruction of the island head usually returns a stable navigation channel with adequate depth. Most bullnoses are used in lock and dam pools. Figure 7.4 shows bullnose structures on the Mississippi River.

FIGURE 7.4 Photograph of bullnose structure on Mississippi River

7.4 HARD POINTS

Hard Points structures are a concentration of stone placed at regular intervals along an eroding bank. These structures are placed to bankfull elevation. The spacing is site specific, but generally does not exceed 500 feet between structures.   St. Louis District constructs Hard Points out of “A” stone (well graded, 5,000 pound max size) with a crown width similar to a stone, spur dike. Hard Points can be keyed in, trenched in, or placed on the bank. When a series of hard points are installed, some continued erosion of the upper bank between the hard points typically occurs, but the structures are able to maintain the overall channel alignment off the riverward end. With hard points, the toe of the bank will generally stop eroding which leads to stability of the bankline and channel alignment, although some minor scalloping of the top bank could still occur. To save rock, some hard points can be built with a sloping crest starting at the top of bank.

Hard Points are an option to full bank protection. If spaced properly, deposition will occur between the structures. This will save the stone which would have been needed to protect the bank between hard points. The scour holes created at the ends of each Hard Point provides beneficial aquatic habitat. Hard Points are typically used in side channels where flows are low and infrastructure is not an issue. Figure 7.5 shows an application of Hard Points.

FIGURE 7.5 Hard Points photograph

CHAPTER 8

CASE STUDIES

8.1 SCOPE

Case studies are provided to show the diversity and evolution of river training systems in use on U.S. navigable waterways. The examples are: The Missouri River, Upper Mississippi wing dams (dikes), the J. Bennett Johnston Waterway (Red River Waterway), the Tennessee-Tombigbee Waterway – Divide Cut, and the Lower Columbia River.

8.2 MISSOURI RIVER

The Missouri River is an example of open river navigation (no locks). The Missouri River, which begins in Montana and empties into the Mississippi River near St. Louis, Missouri, is approximately 1,900 miles long. Since the river was shortened approximately 75 miles during the course of the navigation project, the river mileage had to be revised in 1932 and again in 1960. All locations and lengths of the Missouri River listed in this manual are referenced to the 1960 mileage figures. Omaha District oversees navigation on the upper portion of the river from Sioux City, Iowa, to Rulo, Nebraska, a distance of 234 miles, and Kansas City District oversees navigation on the reach of river from Rulo, Nebraska, to the mouth, a distance of 498.4 miles. Improvement work on the river, consisting of the removal of snags, was first performed by the Corps in 1832. In 1912, authority was granted by Congress to maintain a 6 ft. by 200 ft. navigation channel from Kansas City, Missouri, to the mouth. In 1927 authority was extended to encompass the reach of river from Sioux City to Kansas City for a total navigable length of 734 miles. In 1945 authority was again extended to increase the channel depth to 9 ft and the channel width to 300 ft. Under the 1945 authorization, $190-million was spent on the project. It was completed in 1980.

On the Missouri River, where previous engineers designed an extensive system of dikes and revetments believed necessary to accomplish the desired navigation channel, it was determined over time that considerable modification in this original design was necessary. Therefore, the length, spacing and orientation of individual structures were modified as necessary to improve the effectiveness of the training works. This design “during construction” concept is almost a requirement when working with alluvial streams, and one should not be bound by the perception of a preconceived system that has not been tested up to that point.

With the help of over 3,500 dikes (in the Kansas City District boundaries) the channel is entirely self-scouring. While no locks or dams are located within the Kansas City District, six large dams are located above Omaha, NE, in the Omaha District. Thus the river flow is controlled, except during extreme flow events, from Gavins Point Dam (the last dam in the series) downstream to the confluence with the Platte River, and semi-controlled below that point.

With the dams operational, the risk of flooding caused solely by snowmelt has been significantly reduced and the frequency and stages of other floods have been reduced as well. Flow levels can be augmented with stored water from the reservoirs when natural flows below Gavins Point Dam are too low. Theoretically the reservoirs have the capacity to supplement navigation flows through three years of drought. The navigation season, basically when the river is free of ice, usually runs from 1 April to 1 December.

In 1985, 6.5 million tons of commodities were transported on the river, with upbound and downbound shipments running about even. Commercial sands and gravels accounted for approximately 4 million tons of this total. Other principal upbound products were petroleum, building materials, chemicals, salt, molasses, fertilizers, and steel. Downbound commodities included grain, grain products, tallow, chemicals, and petroleum products.

The standard tow size is four to six barges on the river above Kansas City. Six to nine loaded barges, or as many as twelve empty barges, can make up a tow downstream of Kansas City. This number of barges was cut in half for the low-water years 1988 through 1990.

A typical square-ended barge (also called a box barge) is 200 ft long, 35 ft wide, and 12 ft high. A barge with an angled end (a rake barge) is 195 ft long, 35 ft wide, and 12 ft high. Since the sloping end provides less drag, a rake barge is used as either a lead or trail unit in a tow. A single barge carries approximately 1,500 tons when loaded to the standard 9 ft draft. During periods of high water, barges can be loaded to a draft of 10 or 11 ft.

The normal winter (non-navigation) average stage on the Kansas City gauge is 10 ft (referenced to a gauge zero of 716.4 ft National Geodetic Vertical Datum (NGVD) with an associated flow of 20,000 to 25,000 cubic feet per second (cfs). The normal summer (navigation) target stage, also called the "full service flow," is 13.9 ft on the Kansas City gauge with a flow of 41,000 cfs. Because severe drought gripped this area from 1987 to 1990, flows had to be reduced to below normal levels during 1988, 1989, and 1990. The winter flow was dropped to less than 9 ft and 21,000 cfs at Kansas City, and the navigation flow was reduced to a 12 ft stage with a 38,000 cfs discharge. This low navigation flow is the "minimum service flow." Due to depletion because of the drought, it is estimated that six years of normal rain and runoff will be required to refill the reservoirs back to standard operating levels.

The sand-silt-clay sediment load before construction of the six dams on the upper Missouri River was approximately 200 million tons per year. After the dams were built, the sediment load decreased to 50 million tons per year, with the percentage of silts and clays decreasing and the percentage of sand increasing. No dredging was performed from the time the project was completed (1980) until 1988. Due to the drought and associated low stages, some dredging was performed during 1988 and 1989 (approximately $775,000 each year) and dredging was required in 1990. This work was carried out using a cutterhead type dredge (the *Thompson).*

Early dikes on the Missouri river were constructed of timber piles in a single row, double row, or triple row configuration, and were usually constructed on a willow or lumber mattress. The purpose of the mattress was to reduce scour at the base of the pilings. To aid navigation, a tall pile clump served as a marker at the river end of the dike. A pile clump consisted of three timber piles driven close together and angled slightly (called the batter angle) so that the tops of the piles touched. The piles were then bound together near their crowns with wire rope.

These dikes were very effective in accumulating sediment. As the dikes became buried in sediment (actually becoming part of the bank and then the overbank area), they would be extended into the river, further constricting the channel. However, due to increases in labor costs and the amount of damage inflicted by ice, flood, and other forces, the pile dikes were reinforced with stone.

Since the early 1960's, all dike construction and repair work has employed maximum-weight, 2,000 lb, quarry-run stone. Specifications state that stones smaller than ½ in. cannot exceed 5 percent. This stone is obtained at any of a number of quarries located adjacent to the river.

The “floating plant” method of construction is used almost exclusively. The procedure for this method is as follows: Corps surveyors provide a baseline from which the contractor can establish the work location. A dragline barge is then anchored in position with spuds. Rock is brought to the work site on flat-decked barges and placed using a dragline bucket controlled by the barge-mounted crane. At times a “spacer” box is placed between the material barge and the dragline barge so that rock can be dragged off the material barge and placed accurately. In situations where the dike is above water or in shallow water, stone is placed using a clamshell bucket.

The only structures not built by the floating plant construction method were revetments built parallel to the bank during the winter (when the river was iced over) and some secondary channel closure structures (where the water was too shallow for the construction barges). For this work, rock was hauled overland in trucks and dumped.

The purpose of the dikes was to constrict the width of the river and stabilize the navigation channel. This practice has been going on for a long time, and as a result, many of the dikes now extend hundreds of feet into the present-day banks. Most modern dikes are level-crested and extend 300 to 600 ft into the river. The contraction width is called the “rectified channel width,” varies from approximately 800 ft at Rulo, NE, to 1,100 ft at the mouth of the Missouri River (550 to 750 ft., respectively, in areas with sills). The contraction width is the distance from the river end of the dike to the opposite bank, or the river end of a dike to the river end of a dike directly across from it.

The dikes are built with a specified crown width of 4 ft. The current Construction Reference Plane (CRP), which was developed in the mid-1970’s, revised in 1982, and revised again in 1990, is designed to give a design consistency regarding river training structure heights. Building dikes referenced to the CRP would theoretically have all structures overtopped for the same number of days each year. The CRP is based on the flow that is equal to or exceeded 75 percent of the time (during the navigation season) and takes tributary flow into account as one moves down the river.

River training structure design heights range from -2.0 to +6.0 ft CRP, depending on location and type of structure, with heights increasing as the mouth of the river is approached. Dikes are normally spaced 600 to 1,000 ft apart in the reach from Rulo, NE, to Kansas City, MO, and 800 to 1,000 ft apart from Kansas City to the mouth. Generally speaking, dikes are farther apart on convex bends and closer together in straight reaches. The spacing seems to be a function of the radius of the bend since flatter bends do not exert as much control over the flow and closer spacing is required. When additional dikes are required after the initial construction of a system, these spacing guidelines were not strictly followed. Most dikes are not marked; but in two or three cases where the dikes are a clear hazard to navigation, a pile of rock 20 ft long and 3 to 4 ft. tall is used to mark the river end of the dike. This marker aids a pilot in determining the location of the dike during periods of high water when the dike is submerged. In some areas, low-elevation sills, or sill extensions on existing dikes, are employed. A sill, by the Kansas City District’s definition, is a dike that is submerged more than 95 percent of the time.

Many different types of river control structures are used, including L-head dikes, chute closure dikes, bankheads, convex dikes (located on the inside bank of a bendway), concave dikes (located on the outside bank of a bendway), kicker control structures, crossing control structures (used to tie a kicker control structure to the bank), floodwalls, and low-elevation underwater sills.

The District schedules spring, summer, fall, and almost always, winter inspections. These examinations are staffed by inspectors from the Engineering and Operations Divisions of the District, along with personnel from the Missouri River Division office and the District project office at Napoleon, MO. It is felt that a careful inspection of all river training structures four times a year is sufficient.

Types of dike damage

The following types of dike damage are reported by the District:

- Settling of stone (usually in the first 2 years after a dike is built).
- Flanking
- Loss of rock at the channel end of the dike

Causes of dike damage

The following causes of dike damage are reported by the District:

- Ice and ice bridging
- Floods
- Propwash
- Natural weathering of rock
- General wear and tear
- Towboat impacts (infrequent)

Repair Criteria

The following criteria are used to determine when, or if, a dike is in need of repair, or redesign and reconstruction:

- Integrity of the project
- Adequacy of navigation channel
- Presence of serious bank erosion
- Integrity of individual structures
- Integrity of structure system
- Environmental consequences
- Extent of damage
- Location of the structure
- Type of structure
- Available funding

In some cases these criteria would be weighed equally; in other cases, some items would carry more weight than others.

Specific criteria for required structure repair are as follows:

- Serious bank erosion
- Inadequate navigation channel
- Structure degraded more than 2 feet
- Damaged area more than 100 feet long (Derrick, 1991)

SUMMARY OF DIKES ON MISSOURI RIVER NAVIGATION PROJECT

DISTRICT	DIKES	DIKE TYPE	NUMBER	CHANNEL LENGTH
Kansas City	3,560	Stone	1331	498
		Pile	600	
		Stonefilled Pile	1629	
Omaha	2,810	Stone	1331	232
		Pile	492	
		Stonefilled Pile	987	
TOTAL	6,370			730

Note: Dike data from Derrick (1989)

8.3 UPPER MISSISSIPPI RIVER

As European settlers arrived in the 19th century, large tracts of floodplain forest and prairie were lost to agriculture and other development. The Mississippi River became an increasingly important travel and trade route. Congress authorized a series of navigation improvements to be implemented by the U.S. Army Corps of Engineers. Thousands of wing dams (dikes) and side channel closing dams (dikes) were built to constrict the main channel and increase its depth.

Wing dam construction began in the 1870's during the development of the 4 ½ and later the 6 foot navigation channel projects. Willow fascine mats were used to construct wing dams, closing dams and revetments from the 1880's into the 1920's (see David Bosse print illustrating wing dam construction using the following link: http://www.mvr.usace.army.mil/Bosse/Plates/Plate20.htm). After that time, repairs to structures and new structures have been built entirely of rock. The stream ends of many wing dams in the upper pools remain exposed in the main channel borders, while the landward ends are buried in sediment. They remain "effective" in narrowing and realigning the navigation channel. In the lower pooled reaches and especially in the Mississippi River below the confluence with the Missouri River, wing dams are common and visible above the water line except during floods (WEST 2000). During the 1930's the lock and dam system was built, which established the present day 9 ft channel depth.

There are numerous training structures of various types along the 857 miles Upper Mississippi River (UMR) - located from Minneapolis, MN to the mouth of the Ohio River at Cairo, IL - including 4,322 wing dams (or dikes), 180 bendway weirs and hundreds of revetments. Of these, 1,205 dikes and 177 bendway weirs are located along the 185 miles of open river portion of the UMR, between Cairo and Locks and Dam 27 near St. Louis. The rest of the dikes and bendway weirs are located along the pooled portion of the river upstream of

Locks and Dam 27 . Training works are frequently located in areas that previously required repetitive dredging, which resulted in significant reductions in dredging volumes and frequencies. The majority of dikes were built from 1900 to 1920 and were constructed of stone. Most were angled slightly upstream, although some were normal to the flow.

By the 1970's the Corps was working to improve efficiencies in channel maintenance, and environmental interests were interested in reducing channel dredging and related impacts of dredged material disposal. A recommendation was made in the GREAT II Channel Maintenance Handbook (USACE, 1980b) for the Corps' Rock Island District, to form a permanent committee to evaluate and plan short-term and future needs of training works.

> Great River Environmental Action Team (GREAT) was authorized by Congress in the Water Resources Development Act (WRDA) of 1976. The purpose of GREAT was to develop a total river resource management plan for the entire Upper Mississippi River system and, in particular, to resolve inter-agency disputes relative to the Corps channel maintenance activities. GREAT was developed in three phases; GREAT I included Pools 1 through 10 (St. Paul, MN to Guttenburg Iowa); GREAT II included Pools 11 through 22 (Guttenburg, IA to near Hannibal MO) and GREAT III included the portion of UMR below Locks and Dam 22.

This case history covers the Rock Island District's 314 mile stretch of the Mississippi River from Locks and Dam 10 at River Mike (RM) 614 to Locks and Dam 22 (RM 300). Twelve locks and dams are located on this reach. All dikes within the District are submerged, with the majority cresting at 2 to 3 ft below flat pool. Within the District these submerged dikes are called "wing dams". Approximately 10 percent of the wing dams are marked with a standard channel buoy anchored to a 1 ton concrete block to aid navigation.

The Rock Island District historically placed approximately 50,000 to 100,000 tons of rock each year to repair, modify, and construct wing dams and other structures to maintain the navigation channel.

Newer structures, such as chevron dikes and other innovative, environmentally supporting designs, have been developed recently and are being constructed and monitored to assess their effectiveness to maintain navigation and to determine their habitat value. Older structures have also been redesigned, mostly by notching to increase flow in the dike field and to increase habitat diversity in the dike fields.

The following presents an example of successful channel maintenance using training structures' maintenance and repair along the UMR.

One recommendation in the GREAT II study team was to check the closing dams at Beebe Island (navigation Pool 22) for effectiveness to keep the sediment moving in the main channel. Dredging records for that period showed a significant increase in average annual dredging volumes compared to recent historic volumes.

> Average annual volume (AAV) combines the volume per dredging event and the dredging frequency for consistent analysis. For example, if during one 10 year period, 40,000 cubic yards was dredging every five years, and during the previous 10 year period, 30,000 cubic yards was dredged every two years, the average annual volume over that 20 year period would be 11,500 cubic yards ((40,000

cubic yards x 2 events) + (30,000 cubic yards x 5 events) / 20 years). Using AAV works just as well when analyzing multiple dredging sites within a reach.

Between 1960 and 1990 average annual dredging volume was 30,500 cubic yards, with each dredging event averaging approx 98,000 cubic yards (taken from historic dredging records, Rock Island District). Seven wing dams in the area were repaired in 1994, two of which were extended (one 50 feet and one 475 feet); three of these were notched. Subsequent to this work, dredging requirements have been eliminated. The last dredging event at Beebe Island was in 1993 (70,733 cubic yards). Additionally, there have been no increases in dredging volumes in the dredge cuts immediately downstream (Whitney Island and Turtle Island), covering a five mile area downstream of Beebe Island). Therefore, sediment deposition has been restricted at Beebe Island and the material has been incorporated into the river bedload and travels down river without off-site impacts. Early studies regarding bedload sediment transport and off-site impacts have been performed. For example, since the early 1980's, placement of dredged material in the thalweg has been used successfully at 10 sites along the UMR. The thalweg is the line following the deepest part of the river. Dredged sand is introduced back into the river in deeper areas of the main channel (with depths greater than 20 feet), leaving sensitive habitat regions (side channels, sloughs and shallow backwaters) undisturbed (Stang and Miller 1985). In addition to lower costs and minimal operational problems, effects on habitat resulting from thalweg placement are small (Baker, et al., 1984). This compares with the earlier description of KSM (Keep Sediment Moving). It is important that such studies determine (or at least estimate) the maximum bedload transport capability for a study area, so a project does not cause that portion, or a portion further downstream to become overwhelmed (potentially resulting in increased dredging requirements or off-site impacts to the river environment).

There are many other examples of successful wing dam repairs and upgrades. There are also examples of less successful wing dam repairs and upgrades. One extreme example of moving sediment and overwhelming a system is located in Pool 18, between Keithsburg and Locks and Dam 18, covering a total of 14.5 river miles (RM 413 to 427.5) over a period of 11 years (1987 through 1997). This area shows a history of dike repair resulting in sediment moving downstream - impacting downstream dredging areas rather than the sediments becoming completely incorporated into the sediment bedload. This work was performed during the 1970's and 1980's. Extensive computer modeling was completed prior to construction; primarily 2D modeling. Since that time, computer and physical modeling techniques have improved dramatically (see Section 11). The rise in dredging requirements and the impacts on the channel dynamics within this portion of Pool 18 has caused great concern to the navigation industry. There have been increased instances of groundings in this area due to accelerated shoaling at downstream areas, and great challenges to adequately mark the shoaling areas with buoys. Also, there have been concerns from the environmental community due to increased sediment deposition in environmentally sensitive areas. There are a variety of factors that could have compounded the movement of sediment within this study area, including overbuilding of dikes, occurrence of significant hydrologic events and input from large tributaries (such as Des Moines River), accelerated bankline and island erosion, or natural pool progression (sedimentation issues moving from upper to middle portion of pools). Maintenance records, such as these for Pool 18 clearly show that in some portions of the river, dredging is not going to be eliminated. Training structures and dredged material placement, supplemented with regional sediment management and state-of-the-art modeling techniques will have to be incorporated into the channel maintenance program to ensure a safe, reliable, sustainable and cost effective navigation transportation system. Training structures are a big part of the program, especially in areas such as Pool 18 of the UMR.

Experience and analysis of records shows that wing dam work in Rock Island District typically results in annual dredging reductions of approximately 100,000 cubic yards.

8.4 J. BENNETT JOHNSTON (RED RIVER) WATERWAY IN LOUISIANA

The Red River Waterway Project uses a wide variety of river training structures, which makes it an excellent example of river training techniques. These structures include:

- Stone Dikes
- Stone Revetments
- Articulated Concrete Mats
- Timber Pile Dikes
- Timber Pile Revetments
- Trench Fill Revetments
- Kicker Dikes
- Channel Realignments

The Red River Waterway is the latest waterway project in the U.S. with construction of the locks and dams completed in December 1994. The 9-ft deep by 200-ft wide navigation channel connects Shreveport, LA, to the Mississippi River. The 234.5 mile navigation channel has five locks and dams for a total lift of 141 ft. Extensive river modification and training works were needed for this river that is heavily laden with sediment. There are 177 cumulative miles of revetments and dikes and 16.2 miles of articulate concrete mats. Construction costs were $2.1 billion with channel improvement works (realignments, revetments, dikes) costing $900 million.

The channel work consisted of both realignment and stabilization. Many bends on the Red River were too tight to accommodate commercial navigation by the Red River Waterway channel design tow (3 barges in tandem). Therefore, these bends were realigned using a channel cutoff across the neck of the old bendway. On the Red River, the pilot channel concept for channel realignment was utilized. This technique includes excavating a pilot channel that is smaller than the desired channel section and allowing the natural erosive action of the river to develop the pilot channels to their ultimate section. On the Red River, pilot channels as small as 80 ft wide were excavated that quickly developed to the typical channel width of 450 ft to 600 ft (Pinkard, 1995). The total number of channel realignments constructed on the Red River Waterway was 36 which shortened the river by almost 50 miles (Pinkard 2001a).

<u>Bank Stabilization</u>

The type revetments used on the Red River were trenchfill, stonefill, and timber pile revetments. At those sites where the desired bank line was located landward of the existing bankline, trenchfill revetment was used. This type revetment includes the excavation of a trench along the desired channel alignment and filling the trench with stone. As the bankline continues to erode, the trench is undermined and the stone in the trench launches down the face of the bank. This process stabilizes the bank, thus locking the channel alignment into place. Trenchfill revetments have proved to be very effective on high energy rivers like the Red River that primarily traverse easily erodible soils.

At those sites at which the desired bankline is located riverward of the existing bankline, stonefill revetment or timber pile revetment were used. These type revetments protect the bank by inducing sediment deposition behind the revetment and thus building the bankline out toward the revetment. If the river is shallow along the desired alignment, stonefill revetment is used. This revetment includes placing stone in a peak section or a section with a flat crown along the desired alignment. In the deeper river locations, timber pile revetments were used. Timber pile revetment includes driving timber piles along the desired alignment and placing stone around the toe of the piling. These revetments are used in the deeper sections of river because they are cheaper to construct than are massive stonefill revetments. Once sediment deposition has occurred behind the stonefill or the timber pile revetment, the revetment is raised or "capped-out" by placing additional stone on top of the deposited sediment. Capouts utilize the sediment deposition behind the revetment to reduce the stone required to raise the revetment to its desired level. This method of construction is less expensive than initially constructing the revetment to its ultimate elevation. Due to the heavy suspended sediment load carried by the Red River, sufficient deposition usually occurs behind the revetments to allow the capout to be constructed after one or two high-water seasons. (Pinkard 1995)

Channel Control Structures

On the downstream end of revetments, especially those in the upper reach of each pool where channel depth is most critical, kicker dikes were provided. This type dike is an extension of the revetment and forces the channel crossing from the revetment to the revetment on the opposite bank. Providing these structures helps maintain navigable depths within the channel crossing, thus reducing maintenance dredging. In the very upstream most reach of the pools, Additional Contraction Structures (ACS) is provided. ACS are stone dikes that extend from the convex bank to develop and maintain the channel against a revetment on the opposite bank. By constricting the channel, a narrower, deeper channel that accommodates commercial navigation is maintained. (Pinkard 1995)

Performance

Since the waterway was opened, dredging has been required in the approach channels to the locks and dams and to a lesser extent within the navigation channel within the pools. Modifications to the locks and dams have reduced the required dredging, but still some dredging is required. Within the pools, the Vicksburg District continues to capout revetments and constructs channel control dikes to reduce dredging within the navigation channel. However, given the flow and sediment conditions on the Red River, required dredging to provide and maintain a viable commercial navigation system has been, and continues to be, of manageable quantities. (Pinkard 2001b)

8.5 TENNESSEE-TOMBIGBEE WATERWAY-ALABAMA AND MISSISSIPPI

The Divide Cut project provides for a 31.7 mile navigable channel linking the Tennessee River system with the Tombigbee River system. It is situated in Tishomingo County, MS and generally follows the Mackey's Creek and Yellow Creek valleys. Principal features include a navigation channel with 12 ft depth and 280 ft bottom width and side slopes from less than 10 ft high to 200 ft in height with stone slope protection. The project includes disposal areas covering 5,000 acres and filled with 150-million cubic yards of materials, 5 major and 186 minor drainage structures and 254 relief wells. The normal pool elevation is 414 feet NGVD (see Figure 8.1).

The Divide Cut is so named because it cuts through the topographic divide between the Tennessee River basin in the north and the Tombigbee River basin to the south. It, therefore, is the connecting link between these two river basins from which the Tennessee-Tombigbee Waterway derives its name. Approximately 156-million cubic yards of material were excavated from the cut. This volume represents roughly half of the total required for the entire waterway. At its deepest point through the divide, the maximum cut was 175 ft deep. The average depth of cut is about 50 ft.

FIGURE 8.1 Photograph of Tennessee-Tombigbee Divide Cut

The Divide Cut provides a navigation channel with a minimum bottom width of 280 ft. Side slopes range from 1V on 3H at the deepest cut to 1V on 2H in other areas. A 20 ft wide berm at elevation 422 ft. runs the length of the waterway and serves as an access for maintenance. Additionally, 15 ft berms beginning at elevation 450 ft. and located every 30 ft in elevation thereafter as necessary were also constructed. The bottom of the cut elevation is 396 ft.; normal summer and winter pools are at elevation 414 and 408 ft., respectively.

Due to the erosive nature of materials, special treatment of the waterway slopes was required. Approximately 1.2-million tons of stone riprap underlain by 2.2-million square yards of filter fabric was installed from the 422 ft. berm down to elevation 396 ft. This treatment serves the dual purpose of protecting the erodible sands from barge wave wash and from piping and erosion due to groundwater seepage. Additional measures used to control seepage on the slopes consisted of placing approximately 290,000 tons of stone filter on the slopes above elevation 422 ft. and installing over 53 miles of gravel and pipe subdrains on the slopes to intercept seepage. Vegetation was also quickly established on the slopes above elevation 422 ft. just as it was in the disposal areas. The purpose of the berms described previously above elevation 422 ft. was to provide a break in the slope, thereby limiting the length and velocity of surface water runoff. These berms were provided with pipe drains to carry the water collected on them into the waterway.

Construction of the Divide Cut began in April 1974, with the awarding of a contract for dredging in the Yellow Creek Embayment in the north end. The Divide Cut was officially completed and open for navigation with the completion of Section 4/2A in January 1984. The 1986 periodic inspection report placed the Divide Cut cost of construction at $557,299,300.

Description of Riprap

Machine-placed stone riprap lines the slopes of the waterway from the elevation 422 ft. berm down to elevation 396 ft. below water level where it is keyed into the bank. Filter fabric was placed underneath the riprap. The purpose of the riprap and fabric is to protect the slopes from erosion and wave wash. The majority of the riprap is 18 inches thick, has a maximum size of 356 pounds and is reasonably well graded down to a minimum size of 12 pounds. The median size is from 75 to 105 pounds and 15 percent by weight is smaller than the 12 pound size.

The slopes above elevation 422 ft. were covered with a material more suitable than the underlying soil for the establishment and maintenance of vegetative cover. This covering varies in thickness from place to place from a minimum of about 8 inches to a maximum of about 2 feet. Due to the high degree of erodibility, inherently low fertility and acidic properties of the soils within the project area, maintenance of the vegetative cover is important. An exception to this vegetative covering exists above the 422 ft. berm in the contract section 4 and 2A area (approximate waterway stations 12, 340+00 to 12,938+00).

In this reach, a band of stone overlying filter fabric was placed along both sides of the waterway between elevations 422 and 430 ft. The purpose of this stone was to control slope seepage which was of such quantity that establishment of a vegetative cover was impossible.

Periodic inspection between 1986 and 1999 rated the channel and riprap bank protection in good to remarkably good condition. The maintenance contract for the Divide Cut was $70,000 to $75,000 per year. The major maintenance problem has been vegetation removal, but this has not interfered with the use of the channel for barge traffic.

Channel maintenance dredging has totaled 375,000 cubic yards since 1980, which implies an average annual quantity of 12,000 cubic yards; however, dredging is not scheduled each year.

8.6 LOWER COLUMBIA RIVER, OREGON

8.6.1 HISTORY

From 1885 to 1969, the Portland District of the U.S. Army Corps of Engineers installed pile dikes between the mouth of the Columbia River and Bonneville Dam at river mile (RM) 146. The first pile dike constructed by the District was installed at St. Helena Bar where the controlling depth of the river in the natural state was 12 feet (Hyde, 1963). Other early pile dikes were constructed at Martin Island and Walker Island Bars in 1892 to 1893. The bulk of the present-day dike system was built in the periods 1922 to 1929 and 1931 to 1940 (USACE, 1988b). The major periods of dike construction are summarized below:

- 1885 to 1893 – 3 pile dikes constructed
- 1915 to 1920 – 14 pile dikes constructed
- 1922 to 1929 – 81 pile dikes constructed
- 1931 to 1940 – 79 pile dikes constructed
- 1968 to 1969 – 56 pile dikes constructed

Initially, the natural channel of the lower 110 miles of the Columbia River had controlling depths of 12 to 15 feet at low water with those depths located at several bars. Between 1866 and 1876 some dredging was done to increase the controlling depth. The first work by the Federal government to effect a permanent improvement to the channel was at St. Helena Bar where a longitudinal dike was constructed from the right bank and extended at an angle downstream for a length of approximately 6,200 feet (Hickson, 1961). The early dikes were at widely separated locations and were not constructed under a comprehensive plan. As a result, the dikes were only partially successful in controlling the river location and depth (Hickson 1961). Both permeable and solid, dam-like structures were built; however, the latter were determined to be unusable on the sand foundations prevalent in the Lower Columbia River. Turbulence was induced as water passed over the dam-like dikes and along the outer end resulting in undermining of the foundation (USACE, 1967).

The first comprehensive channel improvements were initiated by the Rivers and Harbors Appropriation Act, which was signed on July 25, 1912. The Act provided for a channel 30 feet deep and 300 feet wide, to be secured and maintained by dredging and construction of pile dike control works. The project document stipulated that "permanent works should be used only to a limited extent and after careful study and observations of each locality and generally after dredging had been found ineffective" (Hickson, 1961). After 3 to 4 years of maintenance dredging, little progress was being made toward establishing a permanent or reliable channel. As a result, a focused program of pile dike construction began in 1916 (Hickson, 1961).

When dredging was not effective at maintaining channel depth, pile dikes were used to reduce the cross-sectional area and produce velocities high enough to prevent sediment from being deposited in the navigation channel. The fundamental application engineering relationship is based on $V=Q/A$, where V is river velocity (ft/s or m/s), Q is the river discharge (ft^3/s or m^3/s), and A is the channel cross-section area (ft^2 or m^2).

From approximately 1916 through 1964, natural river conditions were evaluated and mimicked to define appropriate width controls that would move sediment downstream without causing excess scour. For specified channel widths, adjacent/nearby sections of the river with the same (or similar) radius of curvature where desired navigation depths and widths were being achieved naturally were plotted. Widths were then scaled from hydrographic charts of areas where the river had a natural channel section meeting the requirements as to depth, and scaled widths were then plotted against the distance downstream (Hickson, 1961).

For the 35-foot channel example, the width bank-to-bank for river locations having a naturally stable thalweg of -35 ft Columbia River Datum (CRD) was expressed in the equation presented below:

$$W = w + (D^2/3)$$

Where: W = approximate bank-to-bank width (in feet) needed to attain a stable thalweg of -35 feet Mean Lower Low Water (MLLW) at a distance, D, downstream from the mouth of the Willamette River (at Columbia RM 101)

w = width between river banks at the initial point in the Lower Columbia River (at the mouth of the Willamette River), in feet

D = distance downstream from the initial point (mouth of Willamette River) in miles

This equation was used to aid pile dike design in the upper reaches of the project (Columbia RM 35 to 101), as then measured downstream from the mouth of the Willamette River. The equation is of limited value in the lower estuarine reaches of the Columbia River subject to lower velocities and greater tidal influence (USACE, 1968).

Many of the pile dikes on the lower 110 miles of the Columbia River were installed between 1915 and 1929. Dike construction upstream of Vancouver, WA primarily occurred between 1936 and 1939. These early dikes were constructed in connection with several channel deepening projects. At the end of calendar year 1957, the pile dike system consisted of 221 pile dikes and totaled 219,278 linear feet. During the period 1957 to 1967, 35 new dikes, totaling 17,365 linear feet, and 13 pile dike extensions, totaling 3,380 linear feet, were built to reduce the cost of maintenance dredging (USACE, 1988b). Not all of these dikes are still in use, thus they are not all included in this study. Additional dikes were constructed in support of the 40-foot channel project and to address a number of pile dike specific needs throughout the Lower Columbia River. The Portland District has constructed no new pile dikes since 1969, though some existing dikes have been repaired or rebuilt.

A significant amount of pile dike design evolved from experience gained during the first 75 years of operation of the project (USACE, 1961). Lessons learned and rules of thumb defined during this time period include:

- Serviceability of the structures is not increased by preservative treatment of piling or by the use of galvanized-iron hardware.
- The 2.5 foot spacing specified between piles has been found at most locations to provide optimum effectiveness in control of the flow and in structural strength.
- Cutoff structures need to be set to keep as much of the structure as possible (and particularly the spreaders) continuously wet except during low flow conditions allowing access for construction and repairs.
- Stone sizes for revetment are necessary for protection from velocities up to 12 ft/s and action of drift and debris along the face of the dikes. Experience has shown that smaller stones do not stay in place when piling are damaged and lost.
- Shoals have a tendency to reform at about the same location on a bar and the construction of one pile dike has been known to develop scour over more than a 2,000 foot reach of the navigation channel and to reduce the amount of material that will be deposited to the bar. Hence, series of pile dikes are usually placed not farther than 2,000 feet apart.
- During the construction at Miller Sands, depths were encountered up to 50 feet plus 8 foot tides. On several occasions the pile driving crew reported buoyancy caused some of the piles to float out after being driven. On the basis of the Miller Sands pile dikes (and

others), the preferred maximum depth below the approximate low water line is about 30 feet (USACE, 1969b).

During the mid- to late 1960's, the basic design approach described above was supplemented with physical hydraulic model investigations performed by the U.S. Army Corps of Engineers Waterway Experiment Station in Vicksburg, MS and the movable-bed model at the North Pacific Division Hydraulic Laboratory (USACE, 1970).

Historical dredging records tend to support the assumption that the pile dikes are serving their intended purpose – dredging volumes between Puget Island and Vancouver, WA decreased by approximately one-million cubic yards from 1955 to 1960 when compared to 1950 to 1955. The decrease in dredged material is attributed to the construction and extension of pile dikes in the region (Hickson, 1961). However, regulation of the Columbia River flow regime (post-1974) has also played a major role in the decrease of the channel mobility by reducing peak flows during the June "freshet." The term "freshet" refers to an annual event when snowmelt from the Columbia River drainage increases the flow hydrograph of the Columbia River to an annual peak (sustained) value. Timing of the Columbia River freshet is typically from May to July. The USACE pile dikes in the lower Columbia River had to be designed to perform (without structural failure or negative functional consequences) during the freshet season.

8.6.2 PILE DIKE DESCRIPTION AND CONSTRUCTION MEHTODS EQUIPMENT

Pile dikes are permeable groins, typically constructed of two rows of vertical timber piles located on either side of a horizontal timber spreader. The permeability provided by the pile dikes prevents the formation of a solid, dam-like structure which could create turbulent outer flows that can undermine the structure's foundation. Failure of pile dikes is rarely due to breakage but primarily caused by excessive movement of the foundation (USACE, 1969b). Pile dikes are designed to have a functional life of 50 years (USACE, 1961).

Figures 8.2 and 8.3 show the pertinent design details for pile dikes that have been constructed along the lower Columbia River. Pile dikes are constructed by driving the two rows of vertical timber piles at approximately 2.5-foot centers to refusal or to a specified depth. The vertical timber piles are installed alternatively on each side of a horizontal spreader and each pile is securely bolted to the horizontal timber spreader using a 1-inch un-galvanized machine bolt, and bolting the piles to the horizontal timber spreader. On the Columbia River there have been some variations in the spacing and configuration of piles. These variations, which may include three rows of vertical timber piles with the piling spacing of 2.0 feet in the high energy environment at the mouth of the Columbia River, depend on river location and forces imposed on the structures due to river velocities, tides, and waves. In the Columbia River reach upstream of the confluence with the Willamette River at Portland, OR the energy is lower and the piling spacing is widened to 5 feet. The pilings are driven into well-consolidated Columbia River sand to an average penetration of 15 ft, except 20 ft or to refusal near the outer ends of the dikes. The shore end of each dike is anchored with a two-pile dolphin and the outer end with a ten-pile dolphin, of which three piles are extended above the high water as markers. Two-pile marker dolphins area also placed at approximately 150-ft intervals along the dikes.

It is unknown if previous pile dikes were constructed to a specific standard, but ASTM designation D25-99 (ASTM, 2005) provide guidance for timber pile selection. Present practice specifies pilings to be clean, straight poles of appropriate length, with a minimum tip diameter of

7 inches and butt diameter of 12 to 20 inches. The bases of the dikes (and adjoining river bank area) are protected from the scouring action of water and debris by a stone blanket (enrockment) as shown in Figure 8.2. The stones used within the enrockment are specified to be clean, angular rock have density greater than 150 lb per cubic foot. Stone sizes range between 20 to 400 pounds each for applications where currents are not excessive, and 100 to 600 pounds each for pile dike areas subjected to high currents or debris effects. Stone sizes specified for revetment protection, at the pile dike-to- river bank transition areas, are necessary for protection from water velocities up to 12 ft/sec and the associated action of drift and debris along the face of dikes.

Pile dike design has mostly evolved from past experience gained from pile dikes in adjacent reaches. The initial purpose of pile dikes was to adjust the effective width of the river; thereby reducing shoaling and increasing scour potential, decreasing the amount of annual maintenance dredging, and stabilizing bank lines and locations of dredged material placement. Experience has generally demonstrated that each individual pile dike effectively increases scour and provides bank protection over a river reach approximately 2,000 feet long (USACE, 1961).

The outer marker dolphin consisting of ten or more piles tied together with wire rope including one or more king-piles is constructed by driving the piles as close as possible to each other; thereby providing optimal strength. Timber piles used to construct pile dikes average approximately 30 feet in length. King-piles and outer dolphin piles average approximately 40 feet in length. All piles are typically driven using a barge mounted crane fitted with a vibratory and/or impact hammer. Piles, with the exception of the king-piles and outer marker dolphin, are cut off at an elevation of 8 feet above the CRD. Cutoff elevations of piles are intended to keep the piles and spreader continuously wet (except during low river flow conditions when construction and repair occur) which reduces rot, allows debris to pass harmlessly over the top during high water, and does not impede flood flows (USACE 1968). Figure 8.3 shows the various elements of the pile dike that were noted in the 2010 Columbia River inspection (see Chapter 3 and Section 12.5).

The information of the Columbia River Pile Dike System was extracted from AECOM (2011).

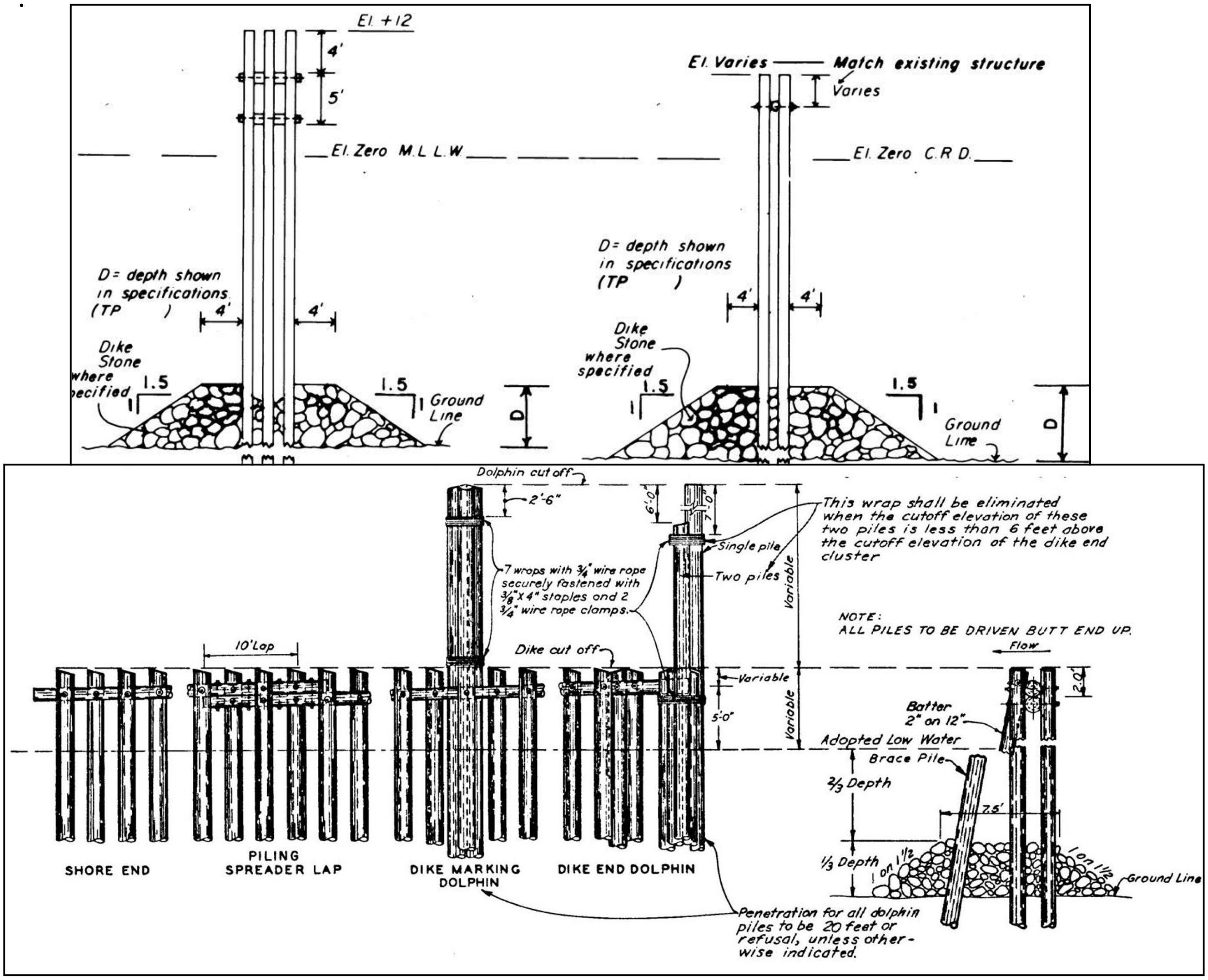


FIGURE 8.2 Typical Cross-Section (top) and Elevation (bottom) for lower Columbia River pile dike

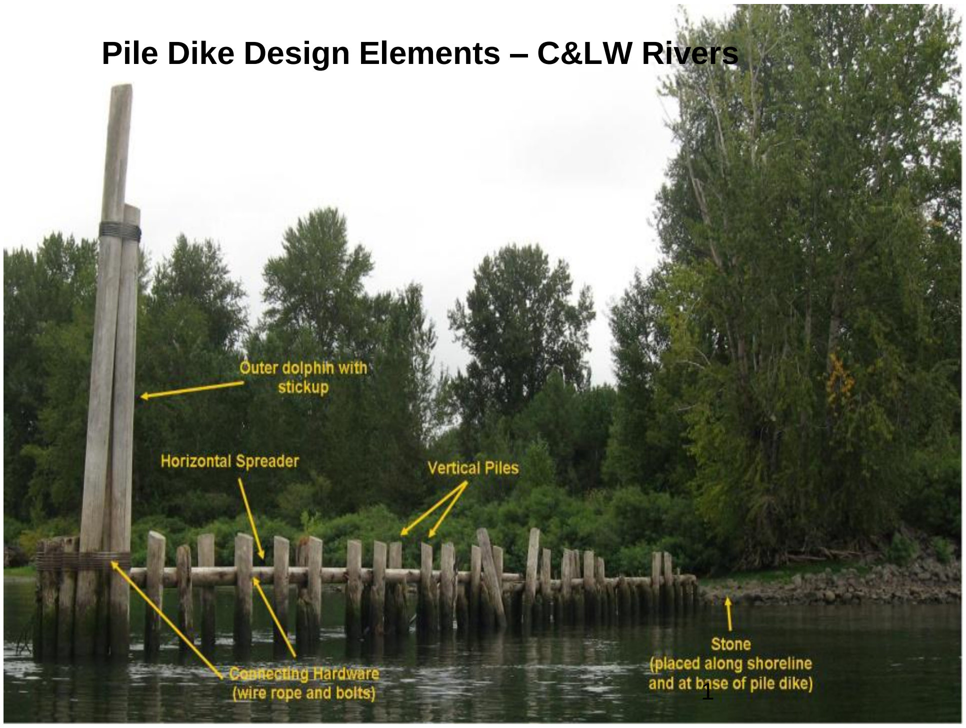


FIGURE 8.3 Pile dike elements noted in inspection

CHAPTER 9

COST

The construction cost of various types of training structures will vary because of many factors, some of which are:

- Availability of material (local or long haul)
- Size of job (mobilization and demobilization)
- Access (water or land)
- Equipment needed
- Labor costs
- Complexity of installation

Following are examples for various U.S. Army Corps of Engineers districts to provide a reference of estimated costs that can be anticipated when designing river training structures.

For the Vicksburg District dike stone was approximately $25 to $30/ton in the winter of 2010. With a minimum dike section of 10 tons per linear foot, a mile of dike would cost between $1.3 and $1.6-million. Dike stone is a "quarry run" stone that requires little or no grading; therefore, it is one of the least expensive stones available. However, the cost of stone construction varies widely within the Mississippi River valley. The cost of stone in the St. Louis District area is significantly less ($16 to $21/ton) than the cost around New Orleans due to the transportation expenses even though the cost of the stone loaded at the quarry is low. Since transportation is linked closely to fuel, the cost of fuel as well as the uncertainty of the fuel market, significantly affects the bid prices of stone. Costs also increase when multiple handling, hauling or stockpiling is required. The least expensive construction method is to place stone from the barge that transported it from the quarry directly onto the dike or revetment being constructed.

Stone placed in a revetment is picked up from the supply barge with a "rock" bucket and spread evenly along the area to be revetted. A rock bucket is a wide mouthed drag bucket with a flat bottom to facilitate picking up the rock from the supply barge. The shape of the bucket and the location of the lift and drag cables also provides for an even distribution of the stone as it is spread onto the area being protected.

More recently large track-hoes have been used on barges instead of draglines for some revetment stone placement depending on the distance from the waters edge to the outer edge of the work. However, the barge mounted track-hoes are used almost exclusively for dike construction. In this work environment the track-hoe and supply barge are lined up parallel to the dike where the stone is cast into place or pulled off the barge into the river where the stone falls through the water onto the dike.

The Rock Island District (USACE, 2006) generated the following cost data for 2005 price levels:

Material	**Cost ($)**	**Description**
Riprap	$22 to $30/ton	In-place, graded, trucked<10 miles
Riprap	$14 to $20/ton	In-place, delivered by floating plant

Bedding	\$16 to \$18/ton	In-place, trucked<10 miles
Bedding	\$12 to \$16/ton	In-place, delivered by floating plant
Sand	\$4/yd^3	Dredged in-place
Fine Gradations of Rock	\$16/ton	
Clay	\$7/yd^3	

The Corps (USACE, 2006) provides the following conversion of cubic yards to tons. As a general rule, once the cubic yards of material are estimated (through Microstation, Inroads, or simple geometry), the following equations can be used to estimate tons of material required:

The conversion of cubic yard volumes to tons of stone follow:

1 cubic yard = 1.6 tons (Rock Island District)

1 cubic yard = 1.5 to 1.6 tons for graded riprap (St. Louis District)

1 cubic yard = 1.6 to 1.7 tons for bedding material (St. Louis District)

Articulated Concrete Mattress (ACM) revetment costs vary widely based on the number of squares being placed in any one season. The basic unit of measure is the "square" (100 square feet) constructed in place. The cost per square is composed of two categories; fixed costs which are independent of the number of squares being placed and the variable costs which are proportional to the amount of work in any one season. An example of fixed costs are plant rental, plant maintenance/replacement, labor and others that must be paid regardless of the number of squares being placed. Variable costs include temporary labor, over-time, fuel, food, and revetment materials whose costs are proportional to the number of squares being used. ACM revetments require 2.5 to 5.0 squares per linear foot of bank being revetted, depending on depth. Therefore, the cost varies with the length or width of the revetment. There are additional costs for stone paving above the ACM from the waters edge to the top bank at approximately 4.6 to 8.5 tons/square. Cost of paving stone is generally higher than the cost of dike stone because it is well graded and requires more time to place.

The cost for Memphis, Vicksburg, and New Orleans Districts are slightly different because the mats are made in three different casting fields. Following is an example of ACM construction and placement cost in the Vicksburg District in 2010:

- Loading and Grading $109
- Towing and Sinking $175
- Mat $115
- Sinking Hardware $ 20
- Gravel $ 18
- Total per square $437

CHAPTER 10

ENVIRONMENTAL DESIGN

10.1 GENERAL

There are several examples of traditional dikes that have been modified. Most modifications are used primarily for environmental benefits. Although traditional dikes are most effective at contracting and deepening the river channel for navigation, it has been found that minor modifications to these structures can increase environmental diversity. These modified structures create a multitude of unique bathymetry and flow patterns around and through the structures without sacrificing efficiency for a safe and dependable navigation channel.

10.2 NOTCHED DIKES

This type of dike has a section removed from the crown, primarily to alter flow and deposition/scour patterns for environmental enhancement. The notch is typically a minimum of 50 feet in width and 5 feet in depth. The location of the notch varies depending on the desired enhancement being developed. This is necessary to minimize the impacts on the river bank. Many times, notches are placed in multiple dikes and in a line to create a secondary channel for environmental enhancement. Figure 10.1 shows the notched dike concept.

On the middle Mississippi River, the St. Louis District notched dikes to help develop habitat diversity within some of the dike fields. The notched dike modifications provided small middle bars within the dike fields with small, chute or side channels on both sides of the bars. Water depths within the dike fields varied from relatively deep scour immediately downstream of the notch to water shallow enough to allow vegetation to grow. This resulted in wide habitat diversity such that a variety of small ecological riverine systems could be maintained within the dike fields. In biological evaluations of the notched dikes in the St. Louis District, it was found that there was a greater “diversity of aquatic macroinvertibrates, and to a lesser extent fish communities.” The District has more than 100 notched dikes on the Mississippi River and the dike notching has shown that channel improvement goals and environmental goals can be successfully combined (Niemi and Strauser, 1991).

On the Mississippi River, the Vicksburg District has provided notches in numerous dikes. These notches were usually not included in the initial dike design but were added later when maintenance was required on the dikes. The location of the notch is usually provided at the lowest section on the dike, wherever that occurs. The size and depth of the notch is site specific depending on how much flow is desired through the notch.

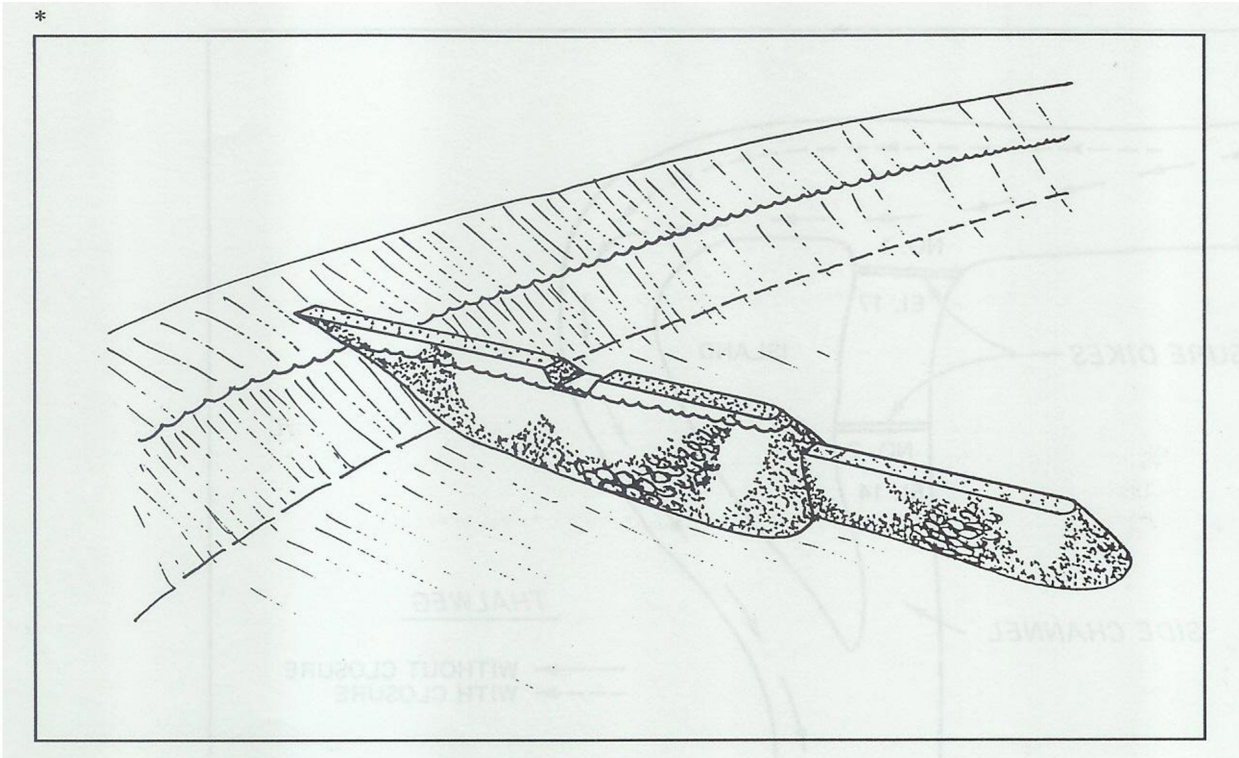


FIGURE 10.1 Dike with notch

10.3 ROOTLESS DIKE

This type of dike has an offset from the river bank, meaning the structure starts some distance off of the bank. The typical offset distance is 100 feet or more. The river bank needs to be revetted a minimum of 100 feet upstream and 200 feet downstream if the bank is not bedrock. A rock pad or sill, minimum 30 inches thick is also commonly used in the width of the rootless section, 100 feet upstream and 200 feet downstream as well. The rootless section provides environmental diversity by altering flow and sediment transportation. Many times, multiple dikes are left rootless and in a line to create a secondary channel for environmental enhancement.

10.4 TRAIL DIKE

This type of dike is commonly referred to as an L-Head Dike and is a downstream extension to a perpendicular dike with the extension parallel to the flow. This is used when large sections of the river need to be constricted. Additionally, an extension can be added both downstream and upstream perpendicular to the flow to form a T-Head Dike.

10.5 W-DIKE

W-Dikes are dikes that have four legs and are shaped like the letter "W," with the apex of two legs facing upstream. The elevation of the legs can be sloped so that the upstream facing apexes are at a lower elevation compared to the downstream portion of the legs. The apexes and varying leg elevation directs flows toward the apexes, forming two scour holes and one depositional bar downstream. The tips of the W-Dikes behave like traditional dike structures, constricting the channel and increasing sediment transport through an area. The landward side of the W-Dike can be attached to the bankline.

10.6 DIKE EXTENSIONS (OFFSET)

Extensions are used when the dike is not performing adequately and additional channel constriction is needed. The offset extension incorporates a gap between the existing structure and new construction, which performs like a notch. The offset can be within the existing dike alignment or can be offset upstream or downstream. These offsets provide a dynamic system for environmental enhancement.

10.7 SLOPING CREST DIKE

These are typical dike structures that change elevation, typically lowest from the tip to highest at the bank (dikes have also been sloped in the reverse for environmental diversity). This allows for different sections of the dike to be engaged at different river elevations. Sloping dikes can also be used while being constructed in depositional areas to mimic the slope of the natural bar. The sloping will reduce the end scour associated with the toe of the structure. Dike crest profiles, including sloping crest dikes, are presented in Section 5.1.4.

10.8 OFF-BANKLINE REVETMENTS

Off-Bankline revetments are used as an environmental enhancement alternative, providing slack water habitat compared to traditional On-Bankline revetments. The Off-Bankline revetment is a row of stone, typically “A” stone, and is placed 5 feet to 15 feet riverside of the existing bankline at an elevation of the existing bank height. Notches are typically left in the revetment to allow fish to access the slack water areas. Off-Bankline revetments are typically constructed in the pools upstream of locks and dams where velocities and depths are lower compared to the open river.

10.9 MULTIPLE ROUNDPOINT STRUCTURES

Multiple Roundpoint Structures (MRS) are alternating rows of rock mounds within the footprint of a typical dike. This structure is built to a two-thirds bankfull stage with the spacing of the rock mounds a function of the structure height. MRS’s are used like a dike to maintain the navigation channel and to create flow and bathymetric diversity within a dike field; therefore, the main benefit of these structures is to create diverse flow and scour patterns for aquatic improvement. Within the structures a series of flow splits form as flow is directed at these structures. MRS structures can also be built in multiple rows with offset roundpoints to create additional flow diversity and habitat. Figure 10.2 shows an example of an MRS on the Mississippi River.

FIGURE 10.2 A double row of Multiple Roundpoint Structures on the Mississippi River, St. Louis District

10.10 ENVIRONMENTAL GUIDELINES AND DESIGN GUIDANCE
(The following is from Burch, et al, 1984)

When providing environmental enhancements and modifications to the design of either new or existing river training structures certain aspects of the details to be included in the design need to be addressed. The designer should consider the general goals that are designed to be obtained by including environmental considerations and fold these goals into the design procedures for the river training structures. General environmental design goals and procedures will be addressed in this section.

General goals

There are several environmental objectives and goals applicable to all dike design and construction:

- Maintain or increase the aquatic habitat diversity by increasing the complexity of physical factors comprising the aquatic habitat.
- Preserve the integrity of existing off-channel aquatic habitat area.
- Schedule construction and maintenance to avoid peak spawning seasons for aquatic biota.
- Design and maintain dike fields to prolong the lifetime of the aquatic habitat (e.g. reduce sediment accretion).
- Maintain abandoned channels connections to the river.

<u>Design procedures</u>

The ultimate configuration of the navigation channel and the locations of dikes and revetments to produce that channel are determined by the river master plan (see Section 4.6). Master plan formulation to ensure incorporation of environmental considerations should include the following steps:

- Formulate a draft river training master plan to achieve navigation, flood control, and bank erosion control objectives.
- Using results of a habitat mapping study, evaluate the existing composition and spatial distribution of riverine habitats.
- Using a multidisciplinary team, set general long-term goals for composition and spatial distribution of aquatic and terrestrial riverine habitats. These goals may be set for major reaches.
- Modify the draft master plan to achieve these goals.

A system of priorities based on anticipated results (in terms of habitat development) should be used to determine which structures should be modified first. The process described above for master plan formulation may result in recommendations to preserve and enhance dike field aquatic habitat. The following steps are suggested for design of a specific dike or dike field:

- Evaluate the long-term potential of the dike field as aquatic habitat.
- Based on the above evaluation, determine whether design modifications or environmental features are in order.
- Consider manipulation of the basic dike design parameters to reduce the elevation of sediment deposition within the dike field.
- Qualitatively project the depths, velocities, and resulting substrates likely to occur in the dike field.
- Consider structural modifications to improve the aquatic habitat within the dike field.
- Consider management techniques to improve aquatic habitat within the dike field after construction.

Environmental features or modifications to dike designs have been applied on the Missouri River and the upper, middle, and lower Mississippi River. Dikes on the Missouri River contain the most environmental modifications and the techniques employed include notches, low-elevation dikes, vane dikes, and minimum maintenance practices. Notches are the most common, over 1,600 having been constructed. Environmental features occurring on the upper Mississippi River are primarily low-elevation dikes and minimum maintenance practices. Notches and low-elevation modifications have been employed on the middle Mississippi River on approximately 75 dikes (64 notches and 11 low elevations); minimum maintenance practices are also used. On the lower Mississippi River, over 200 dikes have been notched or allowed to remain below design grade for environmental reasons.

An extensive study of aquatic habitat improvement to river training structures was conducted by the Upper Mississippi River Restoration Environmental Management Program in the 1980's and 1990's. This study evaluated the following:

- Closure Structures
- Wing Dam Notching
- W-Weirs
- Notched Closure Structures
- L-Head Dikes
- Spur Dikes
- Alternating Dikes
- Stepped Up Dikes
- Bendway Weirs
- Blunt Nosed Chevrons
- Off-Bankline Revetment
- Hard Points in Side Channels
- Vanes
- Cross Vanes and Double Cross Vanes
- J-Hook Dikes
- Multiple Roundpoint Structures
- Environmental Dredging
- Longitudinal Peak Stone Toe Protection
- Wood Pile Structures
- Root Wad Revetment
- Woody Debris
- Boulder Clusters
- Fish Lunkers

The results and recommendations of this study are contained in an Environmental Design Handbook. This handbook was produced by the U.S. Army Corps of Engineers, Rock Island District and is available at the web address given in USACE, 2006.

CHAPTER 11

MODEL STUDIES

11.1 GENERAL

Due to the complex and numerous issues that need to be addressed when designing inland navigation channel training works, the designer often depends on some type of a model study to determine training structure design parameters and details. It is generally accepted that when river reaches are being improved using river training structures that had not been subjected to in-channel stabilization previously, a model study is valuable in determining training structure details. This is particularly true when the planned improvement to a riverine system involves the construction of locks and dams to maintain general water depths, but training structures are required to establish and maintain the navigation channel in the proper location and to minimize shoaling and maintenance dredging requirements.

The types of model studies available to the design engineer are numerous and provide varying types of information/data to use in the design of river training structures and the overall channel improvement effort.

The available types of models include:

- Physical (scale) movable-bed models
- Physical fixed-bed/semi fixed-bed navigation models
- Numerical sedimentation models
- Hydraulic Sediment Response Model
- Tow simulation studies
- Physical and numerical “structures” models

A physical movable-bed model addresses shoaling and scour issues and can be used to determine location and training structure design details (height, spacing, crest profile, etc.) to adequately maintain a satisfactory navigation channel relative to width and depth for anticipated flow conditions. A physical fixed- or semi fixed-bed navigation model can likewise be used to determine training structure location and design details to ensure satisfactory navigation conditions are being designed into the project. A numerical sedimentation model provides similar information/data as a physical movable-bed model but uses computer programs to make the necessary computations to transport the sediment in the model. Hydraulic Sediment Response Models are based on a small-scale (much smaller than physical movable-bed models) physical modeling technology used to replicate the trends of the riverbed. A tow simulator study is similar to a fixed- or semi fixed-bed navigation model but also uses computer programs and visual displays to provide the navigation results. On navigation projects the use of physical or numerical structure-type models are usually limited to details close to the locks, dams, spillways, etc. and normally not the river training structures necessary to maintain a safe and efficient navigation channel alignment. Therefore, physical and numerical structure-type models will not be addressed herein. This chapter will address the various types of models introduced above and provide guidance and references concerning such models.

11.2 PHYSICAL MOVABLE-BED MODELS

Physical movable-bed model studies have a long history in the application of such models to evaluate sedimentation issues relative to navigation projects. Such models have provided exceptional data and information to the engineer in the design and planning of navigation projects, as well as providing useful data and information addressing operational problems on completed projects such as shoaling in lock approaches or addition of a hydropower plant at an existing lock and dam site.

Physical movable-bed models for the design and layout of river training structures normally reproduce several miles (or kilometers) of the river under study. The usual procedure in such a model is to locate the most critical portion of the reach near the center (upstream to downstream) in the model. For controlled waterways such a configuration would locate the lock and dam under study in the center of the model, whereas in open river reaches the greatest area of interest is located in the model center. Virtually the entire river training structure parameters including the channel layout addressed in Chapters 4 and 5 can be studied in a physical movable-bed model. In fact many of the parameters presented were initially studied and developed using physical models before they were used in the prototype. Some innovative or uncommon river training structure types, such as vane dikes and bendway weirs, were first investigated in great detail in the movable-bed model before being installed in various river systems. Those investigations were important in developing design parameters for such structures to ensure that when installed in the prototype the structures would perform satisfactorily and maintain safe and efficient navigation conditions.

The size or scale of physical movable-bed models varies somewhat on the river under study, the type of movable-bed material used, and space or model discharge available. Many riverine movable-bed models have been constructed at scales varying from 1:60 (model to prototype) to 1:500 or greater. Normally movable-bed models at the U.S. Army Corps of Engineers, Engineer Research and Development Center (ERDC, previously the Waterways Experiment Station, WES) were constructed to horizontal scales of 1:100, 1:120, or 1:150. Most ERDC movable-bed models were distorted in that the vertical scale was larger 1:60 or 1:80 depending on the movable-bed material used. At ERDC either fine-grained sand or crushed coal were used on movable-bed models. The sand had a specific gravity of 2.65 with a D_{50} of about 0.25 mm and the coal had a specific gravity of about 1.3 with a D_{50} of 2 to 4 mm. A model constructed using sand was vertically distorted more since the model bed material used was essentially the same as the prototype and velocities to move the model bed material had to be similar to prototype velocities. Often with a sand bed model the horizontal scale may be 1:300 and the vertical scale 1:60. A model constructed using coal did not require as much distortion of the velocity to move the bed material; therefore, such models if it was a reach on a large river such as the Mississippi or Ohio River were often constructed at undistorted scales of 1:100 or 1:120. On smaller projects, such as the Arkansas or Red River the horizontal scale may have been 1:120 and the vertical scale 1:80.

It should be noted that even if a physical movable-bed model is constructed to an undistorted scale, inherent distortions still exist in such models. Typically the bed slope from upstream to downstream is increased to obtain bed material movement. Additionally the velocity scale is generally increased over the theoretical Froude values to obtain bed material movement, particularly on medium to low flows. These distortions or departures from the theoretical values mean that velocities taken on the model cannot be meaningfully associated with prototype velocities. Also, these distortions tend to elevate the stages in the model; therefore, a physical movable-bed model cannot be used to accurately evaluate the impacts of river training structures on river stages. At best the stage readings can only indicate trends (increases or decreases) in stages as a result of the addition of river training structures and the resulting channel bed configuration.

The design, construction, and operation of physical movable-bed models are covered in several very good references which are widely available. The ASCE "Hydraulic Modeling Manual" (see Ettema, et al, 2000) provides an excellent resource on modeling, as does Franco (1978) concerning details to be addressed during physical, movable-bed model studies. Books on similitude, for example Murphy (1950), or river mechanics, for example Julien (2002), also provide excellent information relative to physical movable-bed modeling.

11.3 PHYSICAL FIXED-BED OR SEMI FIXED-BED NAVIGATION MODELS

Physical fixed-bed or semi fixed-bed navigation models, referred to herein as navigation models, are similar in size to physical movable-bed models; however, navigation models essentially address the hydraulics and navigability of a proposed layout and river training structure plan. Since the bed in a navigation model is incapable of scouring, the channel bathymetry is often developed on a movable-bed model and then transferred to the navigation model. At the ERDC there have been several model studies conducted where the study was initially conducted as a movable-bed model and then the same model was transitioned into a semi fixed-bed model to evaluate the navigability of the developed plan (see Franco and Pokrefke, 1983). On many navigation models remote-controlled towboats representing a typical pusher/barge configuration will be operated during the model study to evaluate navigation conditions (see Figure 5.15 for an example of such a study). Such testing is critical when locks and dams are involved. These navigation tests help the designer evaluate the degree of maneuvering required to make such actions as approaching and entering lock approaches and navigating open river reaches through dike fields or over bendway weirs.

As was the case on movable-bed models, with navigation models the most critical portion of the reach is located near the center (upstream to downstream) of the model. For controlled waterways the lock(s) and dam under study would be at or near the center of the model, whereas in open river reaches the greatest area of interest would be located in the center of the model. Navigation models are virtually always constructed to undistorted scales and at ERDC those scales were usually 1:100 or 1:120. Some navigation models at ERDC were constructed to smaller or larger scales, but the vast majority fit into this limited range.

Since a navigation model is physically undistorted and not concerned with sediment movement, all of the Froude modeling laws apply. Therefore, velocities taken off of a navigation model can be converted to prototype values to use in evaluation of project plans tested. Likewise, water surface elevations can be converted to prototype values allowing such model data to be used to evaluate the impacts of river training structures on the water surface elevation and potentially impacting flood heights in the reach. Although navigation models do not necessarily address sediment issues, tracer-type materials, such as plastic beads, walnut shells, or other type materials can be introduced into a navigation model to evaluate potential areas that may tend to shoal with the tested plan in place.

The design, construction, and operation of physical fixed-bed and semi fixed-bed models are also covered in several very good references which are widely available. The ASCE

"Hydraulic Modeling Manual" (Ettema, et al, 2000) provides an excellent resource on modeling, and books on similitude, for example Murphy (1950) also provide excellent information relative to this type of physical modeling.

11.4 NUMERICAL MODEL STUDIES

Over the past few decades, the development and use of numerical hydraulic models has blossomed and many of the issues that previously were only able to be addressed using hand calculations or physical models can now be addressed using numerical models. Numerical models apply numerical techniques such as iteration to solve simplified versions of complex equations of motion, such as the Reynolds-averaged Navier-Stokes equations for conservation of fluid momentum. They produce detailed pictures of flow in rivers, estuaries and coastal waters under past or future conditions, which are then used in design and evaluation of navigation projects. For example, McAnally and Pritchard (1997) examined U.S. Army Corps of Engineers numerical studies of an earthen sill placed in the Mississippi River to limit salt water intrusion and found that the models accurately predicted the interaction of the sill and river flows. Typical models used in navigation studies include U.S. Army Corps of Engineers models such as HEC-RAS, TABS-MD, AdH, and ADCIRC, which are supported by the Surface Water Modeling System (SMS) (ERDC 2011). There is a very wide range of numerical models useful in the design, planning, and operation of navigation projects addressing the hydraulics in one-, two-, and three-dimensions. Certain numerical models also address sedimentation issues and have the additional advantage over physical, movable-bed models in that suspended sediment transport can be addressed numerically while it was virtually impossible (or at best extremely difficult) to do so physically. Numerical models are also useful in addressing selective withdrawal issues from reservoirs, as well as hydrothermal concerns from reservoirs and navigation dam pools. The ASCE "Hydraulic Modeling Manual" (Ettema, et al, 2000) (see page 23 of the Manual) has a section addressing numerical modeling. Figure 11.1 is a portion of a general research, idealized inlet study conducted by the ERDC. The jetties shown could be developed and studied in either a physical or numerical model.

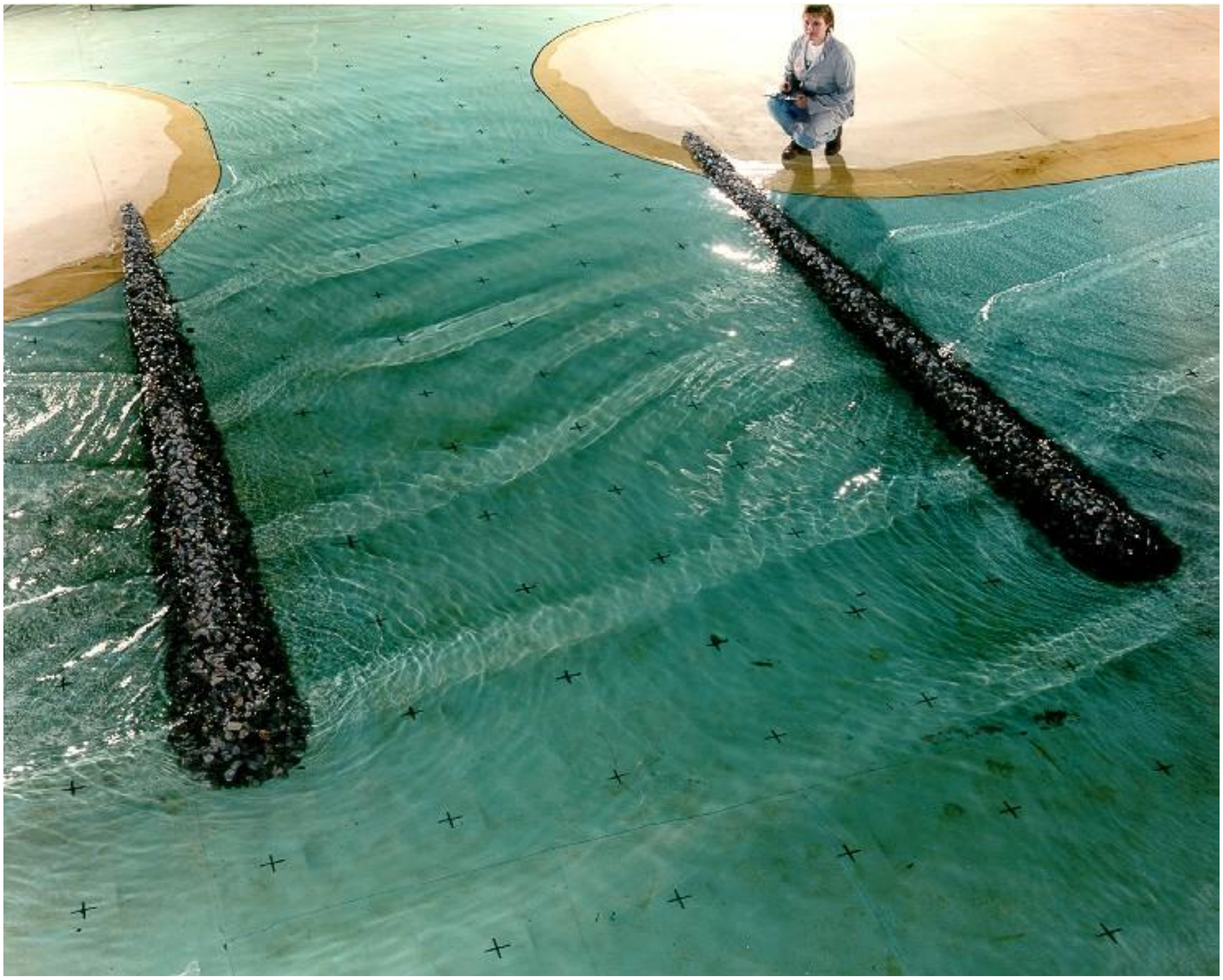


FIGURE 11.1 Idealized model inlet shown in a physical model

11.5 HYDRAULIC SEDIMENT RESPONSE MODELS

In 1993, the U.S. Army Corps of Engineers St. Louis District developed a new small scale physical modeling technology based partially on the larger movable-bed models at ERDC. The smaller scale models utilize scales that allow modeling of a river reach on an area the size of a typical table top (see Figure 11.2). The St. Louis District refined this modeling technique for use in plan formulation and to finalize design parameters utilizing advanced technology and through real-world project implementation results. These models were originally called Micro-Models but have since been renamed Hydraulic Sediment Response Models or HSR Models.

The idea behind HSR models dates back to the late 1800's when Osborne Reynolds used a small-scale model of an estuary. Reynolds' difficulty with modeling on such a small scale was being able to obtain detailed survey information from the bed. Now with the availability of advanced technology, rivers can be modeled and measured on a small scale. HSR modeling is fashioned to many of the principles developed and utilized by Reynolds with the HSR models utilizing scales very small compared to the actual river being studied.

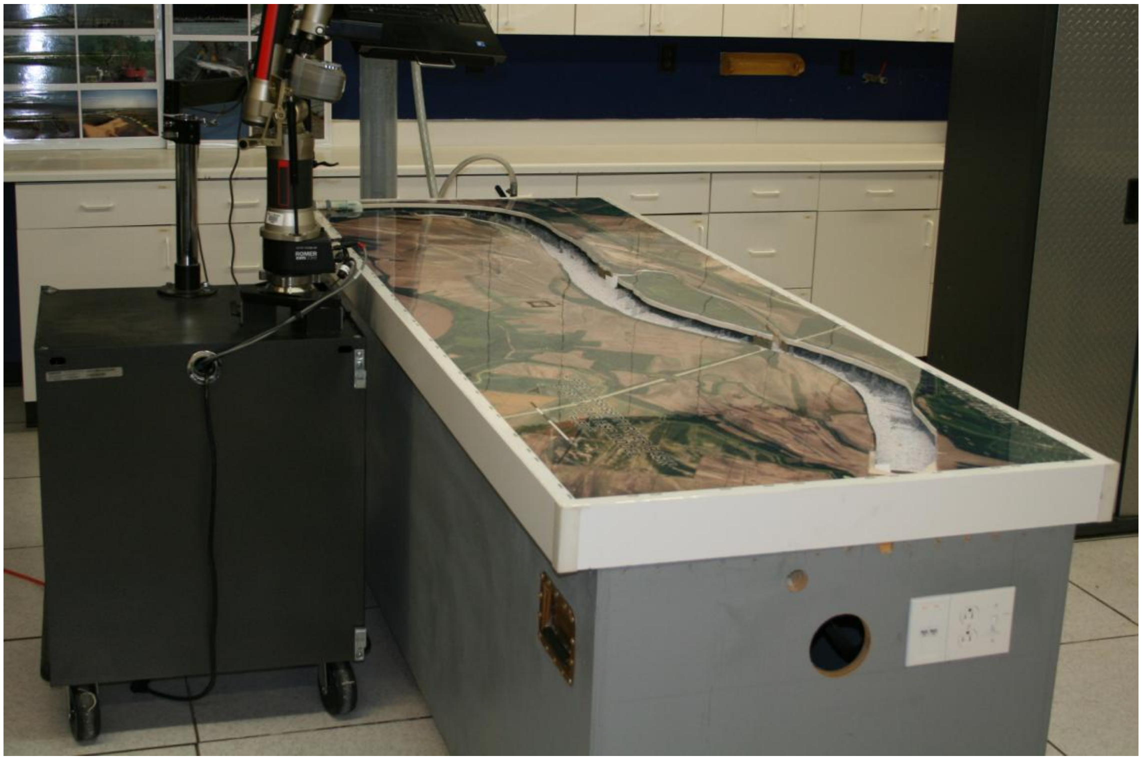


FIGURE 11.2 Photograph of Typical HSR Model at St. Louis District Facility

HSR models use empirical techniques to replicate the trends of the riverbed. These movable-bed models are used for basic river engineering design but do not follow rigid similitude scale ratios established from the Froude and Reynolds laws. Rather, these models focus on producing similarity of the three-dimensional bed response of the model as compared to hydrographic surveys of the river. This similarity is achieved through an empirical calibration process designed to ultimately produce "replication" in which the goal is to achieve a resultant bed configuration in the model that is empirically similar to that of the prototype. Once this similarity is achieved in the model, various river training structures can be added to the model in an attempt to resolve navigation, dredging or environmental issues related to the bathymetry of the river channel being studied.

The HSR models are considered by the Corps' Mississippi Valley Division and other districts to be useful for the study of sediment distribution and riverbed pattern development within a river channel. This includes the general location of bars or depositional areas, scour holes, and the channel thalweg. HSR modeling methods allow for the qualitative prediction of riverbed patterns when applying structure(s) in the river channel. For example, the locations of scour and sediment deposition are desired in most inland waterways when river training structures are implemented to solve a chronic dredging problem.

Dave Gordon, St. Louis District, stated, "The St. Louis District's position is that these models have been used successfully to design and implement many structures on the upper, middle and lower Mississippi River for the entire Mississippi Valley Division. Careful monitoring of projects that were designed using HSR Models has shown that the models accurately predicted the riverbed topography that resulted from the implementation of river training structures in every case. Through numerous research efforts and case studies it has been shown that HSR models have been just as accurate as large scale coal bed models in their ability to reproduce and predict the prototype." However, Maynord (2006) reviewed the HSR literature and concluded that, "Based on the lack of predictive evidence, the micromodel (HSR models) should be limited to demonstration, education, and communication for which it has been useful and should be of value to the profession."

11.6 TOW SIMULATION STUDIES

As the development and use of numerical hydraulic models was taking place, almost simultaneously, vessel simulation capabilities were being developed and applied to navigation projects. The ERDC at WES set up and operates the only U.S. Army Corps of Engineers maritime Ship/Tow Simulator for use in the design of navigation projects for the Corps. The Corps often requires a simulator-based navigation study for modifications to Federally authorized and maintained channels. The ERDC Ship/Tow Simulator can be used to simulate ports or harbors, inland waterways, and other maritime environments. The models used on the simulator accurately produce flow currents, wind and wave conditions, shallow-water effects and bank forces (when applicable), ship handling, ship-to-ship interaction (in a meeting and passing or overtaking and passing situation), fender forces, anchor forces, and tug assistance.

The ERDC Ship/Tow Simulator's primary mission is to aid in the design and evaluation of proposed modifications to Federally authorized navigation projects. The simulator operates in real-time and has the capability to function as either a deep-draft vessel (ship) or a shallow-draft vessel (tow). The simulator hardware consists of a wrap-around animated visual scene display, two radar displays, either a ship or tow console, and a precision navigation display. The two simulators at ERDC can be conducted independently or integrated to represent one virtual world. In an integrated simulation, the pilots controlling the two simulators interact with each other via radio and through the visual scene.

The normal testing procedure is for ERDC researchers to set up and adjust the simulator for the project under study, have actual pilots come to ERDC to verify the simulator study by "navigating" the reach, have the ERDC researchers develop corrective plans, and have another set of pilots come to ERDC to evaluate improvement schemes (see Webb, 1994). Figure 11.3 is a photograph of a one barge tow on the ERDC Ship/Tow Simulator with the control panel in the foreground and the project under study in the background on the display screens.

FIGURE 11.3 Photograph from ERDC Ship/Tow Simulator of one barge tow approaching Morgan City, LA from the South

Most ERDC simulator studies are conducted concurrently with a TABS-MD hydrodynamic model study. Results from the TABS-MD study are used to develop the simulator current databases for the existing and proposed conditions. For riverine simulation studies involving tows, typically current databases are developed for low, medium, and high flow conditions. Usually different visual scene and radar databases must be developed for each flow rate because of changes in land/water boundaries and aids-to-navigation. Low flows have a narrower navigation channel than high flows due to the decreased stage and require different aids-to-navigation systems. For this reason, higher flow rates do not always mean increased navigation difficulties.

While the vast majority of the simulator studies that have been conducted at ERDC have been ship-type studies, there have been a limited number of tow-type simulator studies conducted with river training structures integrated into the proposed channel modifications. Ship/tow simulator studies were conducted on two Mississippi River reaches, Redeye Crossing near Baton Rouge, LA, and Medora Crossing just downstream of Redeye Crossing. Information on the Redeye Crossing study (see McCollum, 1996) is presented herein to provide the river engineer with pertinent study details concerning use of a tow simulator study to address river training structure impacts on navigation.

Redeye Crossing is located about 3 miles (4.8 kilometers) downstream of the I-10 Highway Bridge at Baton Rouge, LA. Traffic in this reach of the Mississippi River includes towboats with drafts up to 15 ft. (4.6 meters) and oceangoing vessels with drafts up to 40 ft. (12.2 meters). Annual dredging at the time of the study amounted to 3-million cubic yards (2.3 million cubic meters), therefore, a river training structure plan using spur dikes was considered for the project to reduce annual channel maintenance costs.

Testing was conducted using the ERDC Ship/Tow Simulator for a variety of deep-draft ships as well as for towboat configurations with 1-, 2-, 4-, 25-, and 49-barge tows. Flow conditions tested varied from a low flow of 228.000 cfs (6,456 cms) to a high flow of 1,500,000 cfs (42,476 cms). Figure 11.4 shows the results of the currents developed using a TABS-MD numerical model in conjunction with a numerical sedimentation study also conducted at ERDC for the Redeye Crossing Reach. Figure 11.5 shows the tracks taken by 25-barge tows during the tow simulator study on the Redeye Crossing Reach.

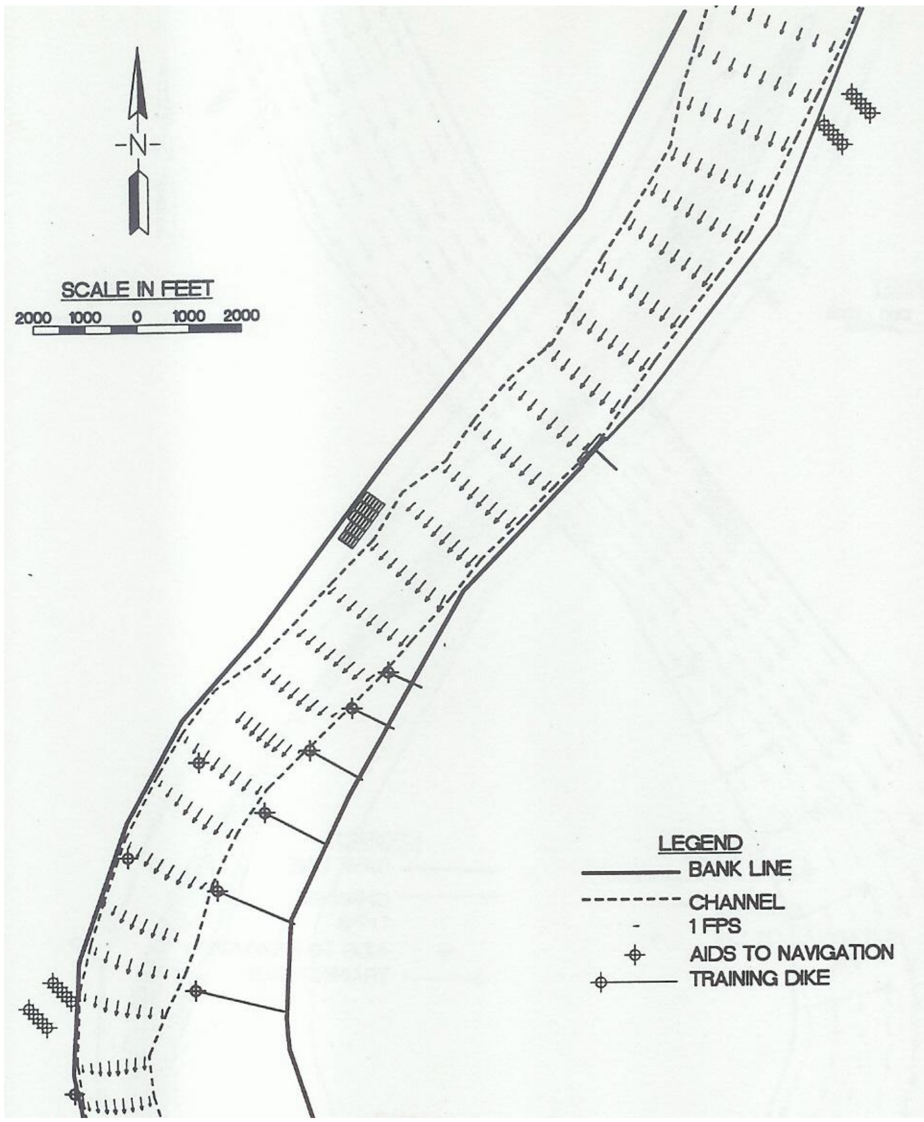


FIGURE 11.4 Currents developed numerically for Redeye Crossing simulator study

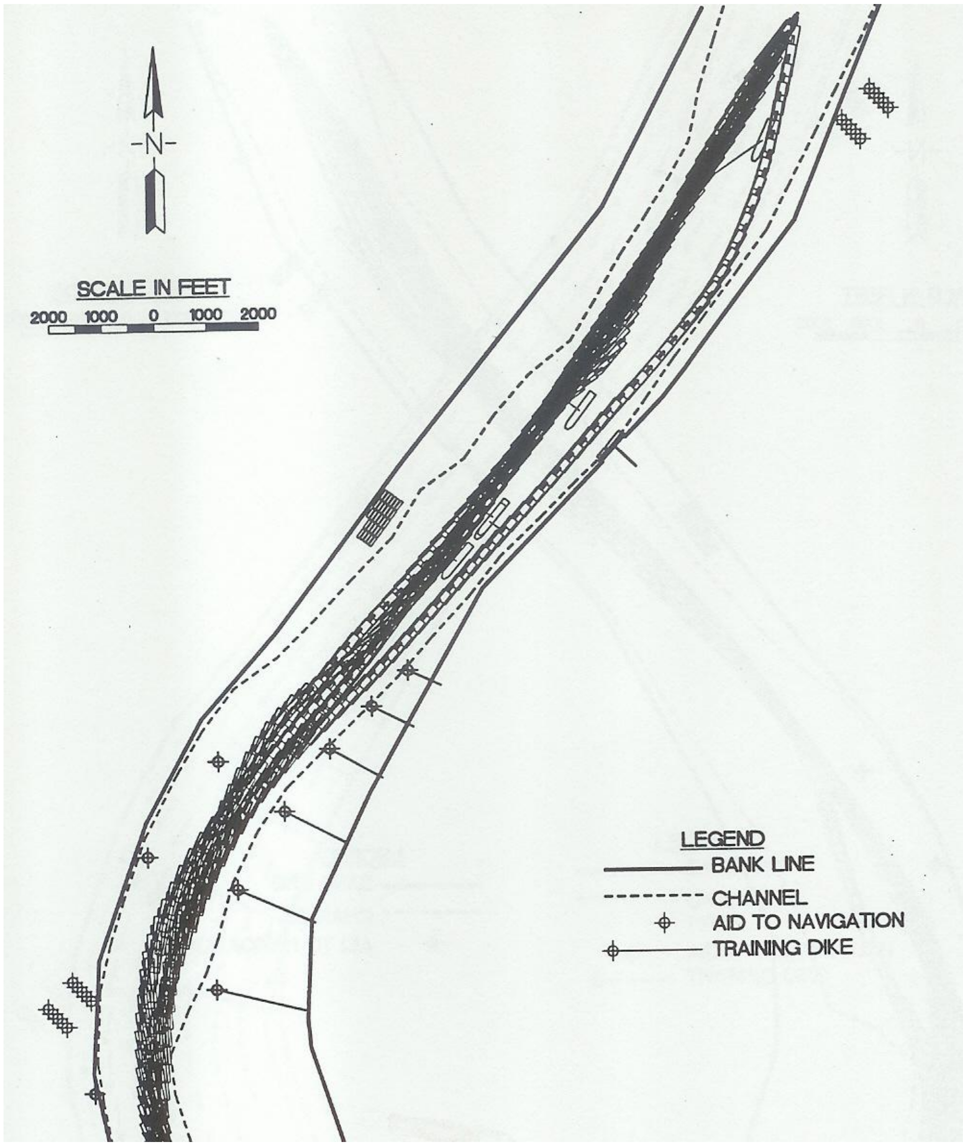


FIGURE 11.5 Tracks of 25-barge tows in Redeye Crossing simulator study

Results of the Redeye Crossing Reach tow simulator study indicated that during certain medium flow conditions with the spur dike improvement plan in place, tow navigation may be slightly more difficult than for existing conditions (no dikes). However, for all other flow conditions the tow navigation conditions with and without the proposed spur dikes were almost identical.

CHAPTER 12

PERFORMANCE, EVALUATION AND INSPECTION

12.1 GENERAL

The true test of any technique to control a river's alignment is how well it performs the job for which it was intended. Dikes are a proven technique and offer the designer a great deal of flexibility. Review of maintenance dredging records and talking with users of the navigation project will help the designer in the evaluation of how well the dikes achieve the desired goal, but the best evaluation of performance is readily visible by periodic field inspections. Items to note during the post-construction field inspection include breakdown or deterioration of construction materials, undercutting of the slope, unusual scour at the landward end or stream end of the dike, changes in the crest elevation, and accumulation of trash. Analysis of hydrographic surveys of the channel reach before and after construction of the training structures will reveal any changes to the channel cross section. Modifications to a dike length or elevation can be made if it is found that such is needed. A slightly under designed structure or series of structures will probably be more cost effective than overbuilding during the initial construction phase. This approach allows time for the river to demonstrate to the designer where and how much additional construction may be necessary in order to accomplish the final channel alignment, channel depth, and channel width.

Inspection methods fall into two categories - underwater and above water as discussed in the following sections.

12.2 HYDROGRAPHIC SURVEYS

Hydrographic surveys of the navigation channel are used to identify shoaling reaches that need dredging. These areas also indicate where training structure repair may be needed. These channel surveys are usually conducted annually during a low flow season. Surveys can be accomplished by traditional depth finders, multi-transducer sweep systems, multi-beam systems, or side scan sonar equipment.

Routine surveys of all dikes are not performed. The only time a dike is surveyed to determine its condition, or even if it still exists, is when repair work on the dike is scheduled. This repair is almost always in an area of the river with a severe sedimentation problem. When a dike is surveyed, the information obtained includes hydrographic surveys of the dike plan, profile, and cross-sections (Derrick, 1991).

The Corps Rock Island District channel survey program consists of government hydrographic surveying crews, operating on the 314 miles of the upper Mississippi River from Lock and Dam 10 in Guttenberg, IA, to Lock and Dam 22, below Hannibal, MO, and on the 271 miles of the Illinois Waterway (IWW) from T. J. O'Brien Lock and Dam, near Chicago, IL to La Grange Lock and Dam, about 80 miles from the mouth of the IWW. Contract survey crews have also been utilized. Each crew operates a survey vessel and utilizes the Global Positioning System (GPS) to obtain horizontal position with a multi-transducer sweep system, multi-beam system, or side scan sonar equipment to obtain depth soundings. A trailerable skiff is also utilized to enable rapid response to reported channel issues and for shallow water survey work.

An average of over 350 detailed hydrographic surveys (condition surveys) are completed each year, representing over 350 linear miles of river. In addition to main channel surveys, non-channel surveys are performed, related to environmental or sedimentation issues, periodic navigation dam surveys for scour analysis, wing dam (dike) surveys, and spring and fall channel reconnaissance surveys.

This operation is much more than merely a hydrographic survey data collection effort. That is, the surveyors are out there as the “eyes and ears” of the channel maintenance program. All of the information gathered, both electronically and cognitively, is used to manage the channel. Field evaluations and decisions are made based on what conditions are like, what kind of alignment exists, what kind of shoaling is developing, how buoys are placed, etc. Input is also obtained from project users (e.g., towboat pilots) on channel conditions to get different perspectives on appropriate navigation recommendations. The District also responds to grounding investigations 24 hours a day, seven days per week.

In the Rock Island District the Channel Maintenance Section and the GIS/Mapping Section have integrated the data collection and data processing procedures to provide a relatively seamless transition from field to office to usable information. Data gathering procedures, base maps, plotting, archiving, GIS database update, and dissemination of information have been standardized.

All 1,048 dikes in the Rock Island District boundaries are submerged (typically 3 feet below the Lock and Dam pools) so shoaling areas are the only indication of dike fields in need of repair. These dike fields are surveyed to determine the extent of repairs needed. Rock Island District historically places 50,000 to 100,000 tons of stone each year to repair, modify and construct wing dams.

The Corps Vicksburg District conducts hydrographic surveys of the underwater portion of dikes and revetments. Survey boats are used to determine depth and width of failure. About 120,000 tons of stone are used for dike repair during a typical year.

The Corps St. Louis District typically utilizes contractors that can be rapidly deployed to collect single beam survey data, which is used to evaluate channel conditions. The engineer specifies the spacing of the transects and is based upon the purpose of the data being collected and the conditions of the river reach being surveyed. The closer the transects are together, the more dense and accurate the data. However, collecting dense data sets require additional time and cost. Surveys of short stretches of river typically use closer transects, while broad channel inspection surveys often use transects spaced farther apart.

In some cases, the District will use 3-line channel patrol surveys to evaluate channel conditions. Rather than running transects, the contractor is instructed to collect data up and down the river longitudinally along 3 separate lines. The 3-line surveys are typically run over lengthy stretches of river (100 miles or more) to get a rapid snapshot of channel conditions. Two of the lines are run following the left and right sides of the channel, usually along the buoy lines or other channel markers. The third line is run down the center of the channel. This data can be collected very rapidly so that the engineers can make a quick inspection of the channel. To reduce processing time, the data is not typically contoured and is usually provided on maps that show the individual elevations referenced to a minimum low flow stage. Soundings that are 9 feet or less are colored red. Soundings that are between 9 and 12 feet are colored yellow. Soundings that are greater than 12 feet are colored blue. Engineers will inspect these surveys upon receipt and quickly determine where there could be problematic areas. The survey crews are then dispatched into those specific areas to perform standard 250-ft spacing transect surveys. The engineers will use these surveys to determine if dredging is needed and to lay out the dredge cut and disposal locations. Engineers determine the frequency and timing of these surveys based upon current flow conditions, short- and long-term forecasts, and from communication with buoy tenders, lock operators, and the towing industry.

The District also performs comprehensive surveys of the entire river channel typically during higher stages so that the surveyors can collect sounding data bank-to-bank and can access side channels and backwater areas. The typical transect spacing is 200 feet but for smaller budgets the transects can be as wide as 500 feet. Due to the amount of river that needs to be surveyed, the District will survey part of the river one year and will survey the other part the following year. These surveys are typically published in paper form and on-line. They are also used for master planning, hydraulic modeling, and design purposes.

When planning for a transect or range line survey of the river, the engineer should use caution when considering adding longitudinal lines to a transect survey to form a grid. Experience has shown that a gridded survey pattern will many times produce erroneous results when using certain types of contouring software. The contours produced can sometimes show unnatural lines that lead to poor results and analysis. It is recommended that only parallel transects be used during data collection.

Condition surveys of specific structures are typically performed during high water using multi-beam equipment. Multi-beam is capable of rapidly collecting dense data over a large area and is useful when the survey vessel can travel over the top of structures. To be collected properly, multi-beam requires skillful operators and significant data editing by trained technicians. Many times the software used to display multi-beam data requires a thinning of the data sets to handle the data without crashing. This will tend to reduce the resolution of the data and sometimes key pieces of information will be lost. Depending on the intended use of the data, sometimes the engineer will have a need for the full un-thinned data set and will use specific software packages designed to handle massive data sets.

When water conditions do not allow the survey vessel to pass over the structure that needs to be surveyed, then a boat-based LIDAR system in combination with multi-beam can be used. The multi-beam will be used to collect data below the water surface and the LIDAR will be used to collect data on the part of the structure that is above the water surface. These data sets can be combined to provide a detailed survey of the structure. The St. Louis District's in-house survey vessels are outfitted with the latest in data gathering technology including multi-beam, LIDAR, and Real Time Kinematic (RTK) capabilities.

In areas where flow and velocity data are desired, then an Acoustic Doppler Current Profiler (ADCP) can be used. ADCP's are typically used by the USGS to collect discharge data. The ADCP's can also be used to calculate and visualize velocity magnitude and direction. ADCP data is typically collected along transect lines perpendicular to the river's current. Single-beam sounding data is usually collected simultaneously so that any analysis can include both velocity and depth data. The ADCP software produces complex data sets. From these data sets, users can extract information such as discharge, velocity magnitude and direction at any point along the transect and at certain intervals within the water column. There are a multitude of customized software packages and add-ons that are available to view and analyze this data. The software the St. Louis District wrote for use in ARC-GIS allows the user to produce plan-view plots of the data showing the velocity magnitude and direction vectors at a user-defined depth, or a plan-view plot of velocity magnitude contours, or cross-sectional plots of individual transects that show velocity magnitude within the water column (isovel contours). The software can also calculate depth averaged velocities and can include fish location data as well.

The collection of any river data requires highly skilled technicians and operators that are properly trained to collect data and set up the equipment to deal with a multitude of variables. Among other things, they should know is - how to compensate for water temperature and the heave, pitch, and roll of the vessel; where the depth sounders are located in relation to the water surface and the GPS; what gages and/or benchmarks should be used to calculate the water surface elevation so that the depths can be translated into the proper elevations using interpolation or other techniques; what the river conditions are and monitor for changes throughout the data collection period; and what datums to use due to the fact that many river gages may not be referenced to the newest datums. This type knowledge, experience and proper equipment setup will give confidence to the engineer that the data received from the field is accurate and precise.

12.3 ABOVE WATER INSPECTIONS

The above water condition of training structures is normally conducted one or more times a year during the low flow season. The Corps St. Louis District conducts these inspections once a year, usually in late summer or early fall, following the spring floods on the Mississippi River. Many inspections are conducted initially by helicopter (see aerial inspection below) using GPS referenced video to capture data that can be used to identify problem areas and damage. The areas identified are then inspected more closely by boat or from the land. Additional assessments are then made using survey data or other means. The Corps Omaha District schedules a boat trip as soon as the Missouri River is ice free, usually late February or early March.

12.4 AERIAL INSPECTIONS

Another method of above water inspection using videotaping the river from the air has proved to be an excellent tool for the purpose of dike inspection. This low-cost technique gives a good overall view and feel for what is happening on the river, but of course, cannot deliver the details. Therefore, to combine the strong points of the aerial videotape with the strengths of an on-site inspection would seem to be an appropriate and prudent course of action. Normally the most important examination is the first low-water inspection of the calendar year. For many Corps Districts this is the first chance to observe the river after winter ice and/or spring high water. Many repair decisions for the upcoming low-water construction season will be based on information gathered during this inspection. Unfortunately the inspectors and engineers making the trip are usually unaware of even the general location or extent of damage that has occurred. Therefore, an aerial videotape shot prior to this inspection trip and studied by the inspectors beforehand would be extremely helpful in preparing the participants on what to expect and at which locations they should concentrate their powers of observation during the tour. This procedure should result in the gathering of more complete and detailed information.

An aerial videotaped fly-over of the river in the late fall would establish an excellent baseline with which to compare the videotape shot in the spring. This comparison would be helpful in establishing when damage had occurred, and benefit in determining cause and effect relationships between river events and observed dike damage. Taping in the fall without leaves on the trees would also help in the evaluation of bank and secondary channel conditions.

For several years the Corps St. Louis District has taken a set of aerial photographs when the ice floes are moving freely on the river as a means of analyzing dike performance. The ice floes act as giant pieces of "confetti" and give a detailed pattern of river flow in and around the dikes and dike fields. These pictures are used to analyze current dike performance and to optimize dike length and dike spacing in any dike design or redesign efforts.

Additional information on this subject is contained in U.S. Army Corps of Engineers Technical Report REMR-HY-8, "Shallow-Draft Training Structure Current Repair Practice and Repair Guidelines," David L. Derrick, April 1991.

12.5 EVALUATION

An example of a combined inspection and evaluation process has been developed by the Portland District for the lower Columbia River navigation channel. This channel is 138 miles long and extends from the Pacific Ocean to the Bonneville Lock and Dam. At one time, 285 pile dikes existed (according to historic records), however the 2010 inspection could locate only 233 dikes. Construction of these dikes started in1885, so the remaining dikes are in various states of deterioration. Figure 12.1 shows the types of pile structures in the lower Columbia channel identified during the 2010 inspection.

The Portland District process is to determine each dike's present condition, hydraulic performance, and fish habitat value. The assessment will be the basis for recommending repair, removal, or letting a dike deteriorate.

This assessment and performance evaluation included:

- Visual inspection: length, height, missing parts, and condition.
- Historic changes using photos from 1939, 1993, and 2009, dredging and historic flow records.
- Models to evaluate hydraulic and environmental performance.

FIGURE 12,1 Types of pile structures on the lower Columbia River channel

The ERDC's Adaptive Hydraulic (AdH) multi-dimensional model will be used to evaluate potential impacts of the dikes on hydrodynamics and morphology of the adjacent areas. The AdH output including the hydrodynamics and bed shear stresses will be used as input to an Eulerian-Lagranian fish behavior model to determine pile dike impact on fish habit.

The 2010 inspection found significant system-wide deterioration of individual pile dikes. However, most of the pile dikes are still achieving their originally intended purpose, and are reducing maintenance dredging and protecting dredged material disposal sites. Pile dikes were found to be creating or protecting juvenile salmon habitat (shallow water morphology less than 18 ft nominal depth) at numerous locations. About 72% of the pile dikes are recommended for retention and repair, 23% are to be monitored or optionally removed, and 5% were selected for additional study.

CHAPTER 13

REPAIR TECHNIQUES

13.1 GENERAL

All dikes, regardless of the specific design or construction material used, will require periodic maintenance in order to ensure that their structural integrity and intended purpose are maintained. The amount of maintenance needed varies considerably, depending upon their location in the bendway, frequency of overtopping, freeze-thaw history, material used, age of structure, and in general what is expected of the structure. The two most critical areas needing maintenance are the root or key (landward end) and the stream end of the dike. In addition, dikes that are frequently overtopped will periodically require repair of the crest of the structure and downstream toe section. Dikes that have been in place for a number of years and were adequately maintained throughout the life of the structure seem to become less prone to damage, particularly those that are part of a system of dikes that become filled with sediment and vegetation.

Revetments also require periodic maintenance to ensure channel alignment and bank stability. The need for revetment maintenance is typically caused by scour occurring along the toe of the revetment which leads to undermining of the upper bank or saturation of the banks during high water which causes the banks to fail when the water recedes quickly (rapid drawdown failure). Other revetment damage is caused by barges being pushed into the banks, wave wash, failure of ACM anchors or cables, trees growing through the revetment and other natural or manmade acts. Depending on the severity of erosion, revetment maintenance is performed by either overlaying existing revetment in damaged areas (reinforcement), creating new bank alignment by grading and tying new revetment into stable banks upstream and downstream of the damaged areas, or patching small areas with riprap.

13.2 EXAMPLES OF REPAIRS AND REPAIR TECHNIQUES

A U.S. Army Corps of Engineers research project in the 1980's looked at repair of navigation project structures, including river training structures. This program entitled "Repair, Evaluation, Maintenance, and Rehabilitation Research Program," evaluated repair techniques at all the Corps Districts that have river training works. A summary report (Derrick, 1991) presented the findings which are presented as follows:

<u>Rock Island District Repairs and Repair Techniques</u>

Legislation for implementing the 9-foot channel project on the Upper Mississippi River contained the authority required to upgrade the existing 6-foot navigation project including the wing dam and closure dam system. As the navigation dams were being constructed, work was underway to modify the height and length of the existing 6-foot channel wing dikes and dams. This work continued through the complete development of the 9-foot project to secure adequate widths, depths and alignment, as authorized, and continues today.

Since the project's construction, periodic repairs to these dikes and wing dams have been performed on a regular basis (nearly annually until the recent past, due to funding constraints). These activities are based on dredging history and condition surveys of the river training structures. The Rock Island District historically placed approximately 50,000 to 100,000 tons of rock each year to repair, modify, and construct wing dams and other structures to maintain the navigation channel.

Since many of the 6-foot channel training works had not been adjusted to the water levels of the 9-foot channel, these structures are lower than originally designed, relative to water surface, and are less effective in maintaining the channel. Therefore, much work today involves modifying these structures to a somewhat higher elevation to help achieve design effectiveness. The intent is to make subtle changes in structures to avoid over-regulating the river environment.

St. Louis District Repairs and Repair Techniques

In the Corps St. Louis District typically 210,000 to 270,000 tons of maximum-weight, 5,000 lb Graded Stone A is used for dike repairs each year. Cost in place for this grade of stone runs from $5.00 to $6.50 per ton. The stone is obtained from the closest of any of a number of quarries located adjacent to the river. Between 50 and 75 dikes are repaired in a typical year. Private contractors perform all work; no dike repairs are performed by the Corps. Typically three dike repair contracts are let, each covering a 100 mile reach of the Mississippi River. The length of a repair contract is 365 days, which allows the contractor to pick the river conditions that are the easiest and most economical in which to work. The contractors used are very experienced in river work and are familiar with the guidelines and techniques used by the Corps.

The contractors use barge-mounted cranes with drag or clamshell type buckets for placing stone. Flat-deck material barges are used to haul the stone. They are generally loaded by trucks that back over ramps onto the barges. At the repair site the contractor is sometimes able to locate the equipment so that the stone can be dragged over the edge of the barge onto the dike. In cases where they cannot locate over the dike (flanking or repairs above the waterline), a clamshell type bucket is used for rock placement. The life of a stone dike is long, and in many cases the time between repairs is 20 to 25 years.

Construction inspection is critical in the St. Louis District. If substandard stone or stone with too many fines is used, the dike will deteriorate rapidly. All dike repair inspections are performed by experienced Corps personnel.

Vicksburg District Repairs and Repair Techniques

Two types of repair contracts are let in the Corps Vicksburg District - major and minor. In 1991 approximately 120,000 tons of rock was used for dike repairs.

Approximately $1-million per year was spent on major repair contracts in 1991. With this type of contract the repair sites and amounts of stone were detailed and specified. Stone costs varied from $7 to $9 per ton (1991 prices), and was barged in from quarries located in Kentucky and Missouri.

Funding for the minor dike repair contract generally ran about $150,000 per year. Cost in place was from $11 to $14 per ton (1991 prices). This was an open-ended contract, wherein the contractor could be directed, depending on need, to work anywhere on the river within the District limits.

A stone repairs contract is awarded annually in the District to repair both dikes and revetments. Approximately 25,000 tons of “A” stone are used for dike repairs and approximately 70,000 tons of “C” stone are used for revetment and overbank scour repairs during a typical year (2006 through 2010). “C” stone is used as traditional stone bank paving and/or as longitudinal peaked stone toe revetments with tie backs to repair areas with overbank scour. During 2009 when stimulus funds were available, 158,000 tons of “A” stone were placed for dike repairs. Both “A” and “C” stone is well graded with “A” stone having a maximum weight of individual stones of 5,000 pounds while “C” stone is much smaller with a maximum weight of 400 pounds.

An average of approximately $450,000 per year (2006 through 2008 and 2010), was spent on dike repairs. During 2009, approximately $4,500,000 was spent on dike repairs. An average of approximately $1,900,000 per year (2006 through 2010) was spent on revetment repairs. The stone repairs funds continue to be expended through an open-ended contract. A river inspection is coordinated annually to identify and document the needed repair sites. The contractor is provided documentation to indicate dike and revetment repair sites, amounts of stone and description of work. Since 2006, the cost for “A” stone in place has varied from $25 to $32 per ton. For “C” stone, the in place cost has varied from $30 to $32 per ton. Stone used on the Mississippi River in the District is now barged to the sites from quarries located in Arkansas, Kentucky, Illinois and Missouri.

The life of a dike generally depends on the following:

- The river’s unpredictable behavior
- Major floods
- Riverflow
- Dike configuration
- Navigation operations (usually minimal damage)
- Ice flows

Omaha District Repairs and Repair Techniques

In the Corps Omaha District the operation and maintenance costs range from $1.5-million to $2.8-million annually. During a normal year, about 50,000 tons of quarry-run, 2,000 lb maximum-weight stone is used for dike repair. Cost in place for this stone runs from $9.50 to $11.00 per ton; therefore, dike repair costs run approximately $500,000 per year, with the number of dikes repaired ranging from 100 to 200. In a flood year, such as 1984 for example, twice that amount of repair work may be necessary. Private contracts are let for most of the work, although as much as 10,000 tons of stone may be placed using District crews and equipment. The Corps usually performs work where small volumes, long distances, or both, make the work unattractive to contractors.

Dike damage is usually observed during the spring, low-water inspection tour. Contracts are let to repair the dikes in early August, and repairs are performed from August through October. If an unusually large amount of repair work is required, the contract can stretch from August to the end of the navigation season (mid-November). Typically two repair contracts are let for the entire District, with the dividing point at Omaha (RM 627). However this policy has not held true recently. Due to the drought, a lack of ice damage, and other factors, structure damage has been minimal. The reach upstream of Omaha has not had a repair contract since

1985, and downstream of Omaha no contract has been let since 1987. The Corps was able to perform all needed repair work in-house personnel and equipment during those years.

The maintenance program is continuous, with the life expectancy of most dike repairs averaging 10 years.

Portland District Repairs and Repair Techniques

Maintenance of the pile dikes in the Portland District involves replacing and/or removing piles. Reference Figures 8.2 and 8.3 for pile dike elements. Minimum over-water equipment includes a crane, barges, and a tug. Necessary upland support equipment includes machinery (e.g., front-end loaders) to load and unload barges and trucks to transport raw materials and debris.

The over-water crane and work barge need to be of an appropriate size and equipped with a vibratory or impact hammer to drive piles and a bucket to pull piles and to dredge stone. All construction is water-based, requiring equipment and crews to work from barges. At a minimum, two barges are needed to base equipment and to place construction debris, as well as to stage raw materials. A tug is needed to mobilize equipment, haul away debris, and maneuver barges on-site (USACE, 2010).

Pile dike repairs occur generally in the following order:

1. Remove damaged or rotten piles by using the clamshell bucket to pull the piles. Alternatively, if pulling is unsuccessful, the pile may be cut at the mud line or top of the stone encroachment. Depending on pile dike condition, it may be necessary to first cut the spreader to separate the piles for safe removal.
2. Remove remnant stone blanket by clamshell dredging at all locations in which timber piles are to be installed. Barge dredged rock and associated sediment to upland transfer area for subsequent disposal or reuse.
3. Drive new timber piles to a depth of approximately 18 feet below the sediment or until refusal using a vibratory or impact hammer. Piles need to be spaced at approximately 2.5 feet on center in alternating and offset fashion to allow for the installation of the horizontal spreader through the center of these piles. In addition, drive the new timber king-piles/outer dolphin piles adjacent to one another to provide increased structural support.
4. Secure piles in place by installing a horizontal spreader and attaching it firmly to each pile with one pieces of steel hardware at each connection point (i.e., one piece of steel hardware consists of a 1-inch diameter length of all-thread bolt, 2 nuts, and a spike grid placed between the vertical pile and horizontal spreader). Secure the outer dolphin and king-piles using 5 pieces of hardware without the spike grid and lashed together using 0.75-inch wire rope. After the nuts are tightened, the threads should be marred or the nuts welded to prevent the nut from working loose and weakening the connection.
5. Place stone around the outer dolphin and along the length of the pile dike, as needed, to prevent scour from undermining the pile dike (AECOM, 2011).

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APPENDIX A

TERMINOLOGY

Articulated Concrete Mattress Revetment (ACM): the current design consists of 16 concrete blocks approximately 17.75 inches by 48 inches by 3 inches cast together on a stainless steel wire mesh (or fabric) that forms a basic unit referred to as a "Square" (4 feet by 25 feet =100 sq. ft.). The wire mesh is located in the neutral axis and is used to connect adjacent squares to each other and to anchors along the bank. The wire mesh serves as a flexible skeleton allowing the assembled mattress to conform to the uneven contours of the stream bed and bank, this flexibility is called articulation.

Bendway Weirs: submerged sills constructed in the navigation channel (within the channel control lines) angled upstream. The structures direct flow toward the point bar, eroding it, to use the river energy to widen the navigation channel. Used primarily in short radius bends where channel width is insufficient for safe navigation.

Board Mattress Revetment: a mat of boards woven together in a square weave with the warp boards, termed the weavers, parallel to the bank, and the weft, designated the weaving or mattress boards, at right angles thereto. Board mattresses require ballast or heavy objects such as stone or concrete rubble to sink and hold the mattress in place.

Closure Dike: structure designed to close a secondary channel and increase the hydraulic energy in the main channel.

Dike: structure used to redirect or retard flow and encourage sediment deposition. These structures can be made of stone or wood and used alone or in groups called a "dike field". They are constructed approximately perpendicular to the bankline to realign a river reach, constrict a channel to increase depth, cutoff side channels and chutes, and concentrate the flow. Other terms used for such structures are dyke, contracting dike, transverse dike, cross dike, spur dike, spur dam, wing dam, groyne, retard, jetty, sill, spur, and wing dam.

Groin (or groyne) dikes: when not used as dikes defined above, but are stone structures laid on a low caving bank (usually behind a sand bar) or in a small stream that act as hard-points to stabilize a caving bank. The groins are spaced depending on the size of the stream and/or the radius of the bend (small stream) and the designer should allow for some continued caving between structures until bank equilibrium and revegetation is reached.

Kicker Dike: provided on the downstream end of revetments to constrict the downstream crossing to eliminate sediment deposition and associated maintenance dredging in the crossing.

L-Head Dike: spur dike extended downstream parallel to flow from it riverward terminus. This is done to extend the influence of the dike downstream at a lower cost than constructing another spur dike.

Longitudinal Dikes: continuous structures extending downstream from a bank, an island or transverse dike generally parallel to the alignment of the channel being developed. Used to establish a desired alignment riverward of the existing bank line.

Quarry-Run Stone: stone classification developed to provide quality stone at minimal costs by eliminating or significantly reducing the need for processing. Initially, the gradations A, B, and C were produced based on varied blasting techniques in the quarry and subsequently loaded directly from the quarry floor to the transportation barges. These gradations allowed a higher percentage of fines as a trade-off for reduced cost and processing.

Revetment: an armament or protective covering used to prevent or minimize the erosion of the stream bed and or bank material (underlayment).

Sills: low-level dikes, usually submerged at all river stages, to encourage navigation channel widening or constructed in the approaches immediately upstream of lock walls to reduce velocities in the approach

Stone Spur Dike: impermeable-type dike composed entirely of quarry-run stone usually constructed normal to the flow of the river or at an angle.

Timber Pile Dike: permeable-type dike constructed using single piles in line, single piles staggered, to single row clumps and multiple row clumps.

Trenchfill Revetment: quarry-run or partially graded stone placed in a trench excavated along a desired alignment so that the stone will slide or roll into the adjacent eroded channel as the bankline is scoured by natural hydraulic forces

Vane Dikes: segments of dikes placed at a slight angle to the direction of flow located riverward from the existing bank with gaps between the dikes.

APPENDIX B

DEVELOPMENT OF CHANNEL CONTRACTION WIDTHS

Paper by Lamb and Ethridge, “Sediment Management on the Mississippi” presented at Fifth Federal Interagency Sedimentation Conference in Las Vegas, Nevada in 1991.

ABSTRACT

The Mississippi River is the fourth largest drainage basin in the world, has a mean average discharge of 600,000 cfs, and carries in excess of 200,000,000 tons per year of suspended sediment. Managing the downstream movement of this volume of water and sediment in such a way that contributes to the project purposes of flood control and navigation is the real challenge. The science of River Engineering is made difficult by the number and complication of the parameters involved when attempting to define this movement by mathematical terms and physical formula. For this reason, it is very important that the actual river responses which incorporate all parameters be analyzed and that empirical relationships based on proven theory be developed for use in design. This paper uses this approach to develop two reach specific design curves for designing stone dike contraction structures based on the same basic concepts. The first curve is for the Mississippi River and Tributaries 9' draft project (Cairo, Illinois, to Baton Rouge, Louisiana, reach) and relates the width of the main channel to the conveyance ($Ad^{2/3}$) allowed outside of the main channel. The second curve is for the Baton Rouge to Gulf project (Mile 181 to Baton Rouge Reach) and relates to highest point in the navigation channel crossing to the conveyance allowed outside of the main channel. Development of these curves gives the engineer a mathematical way of computing required dike height and length to ensure economical project development.

INTRODUCTION

Purpose and Scope

This report presents empirical relationships between channel conveyance and width or depth which are being used to design stone dikes to efficiently move water and sediment down the Mississippi River in a manner compatible with project purposes. Two design curves using the same basic concepts will be presented. The first curve is being used on the Mississippi River and Tributaries (MR&T) project (9' draft), and the second on the Baton Rouge to the Gulf project (40' increasing to 45' draft). The design curves are river reach specific; however, the concepts are valid to both applications and similar relationships can be developed for any alluvial river system.

Mississippi River General

The Mississippi River, which drains 41% of the continental United States, is the 4th largest drainage basin in the world, has a mean annual discharge of 600,000 cfs, and carries in excess of 200,000,000 tons per year of sediment in suspension past the latitude of Old River, Louisiana. In addition to the suspended load, it is not unusual to observe 5 to 20-foot sand waves moving along the channel bottom. Sand waves as high as 40 feet have been noted. Managing this sediment movement in a way that compliments development of the system for flood control and navigation is the real challenge.

Need for Empirical Study

The science of River Engineering is made difficult by the number and complications of the parameters involved when attempting to define the movement of water and sediment with mathematical terms and physical formula. Physical model studies give useful design guidance, and good progress is being made in developing improved two- and three-dimension mathematical models. However, physical models are very expensive; and the varying physical parameters effecting the movement of water and sediment downstream along different geometric alignments through various soils with innumerable combinations of hydrographs make this a very difficult problem to define with precise mathematics. For these reasons, when designing river control works, it is imperative that the river itself be analyzed and empirical relationships developed for use in design. It should be noted that the results observed on the river do in fact incorporate all variables to an infinite degree. These empirical relationships represent valid engineering theory and can be explained using classical hydraulic formulas and observed river responses. As already noted, the specific empirical relationships developed for the Mississippi River are not directly transferable to other systems; however, the concepts and relationships can be used to develop similar data for use in design of other projects.

CONVEYANCE

Definition

For the purpose of this analysis conveyance will be defined as $Ad^{2/3}$ below a flowline equal to the Low Water Reference Plane (LWRP) +25' to +30' (depending upon reach location) where A is the area of the section or subsection and d is the depth. LWRP+25' to +30' was selected as approximately a bankfull elevation which represents a discharge with the most dominant channel shaping characteristics. This selection is reinforced by previous conveyance studies by Anding (1970), which showed consistency in conveyance between bendways and crossings for this discharge and sediment studies by Biedenharn (1987) which show that a preponderance of the total sediment is transported by bankfull or below stages. The term "$Ad^{2/3}$" is easily derived from Manning's flow formula which is classically written as:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

V = Velocity
n = roughness
R = Hydraulic Radius
S = Slope

By substituting into and adjusting this formula for large river sections it is typically written as:

$$Q = \frac{1.486}{n} Ad^{2/3} S^{1/2}$$

Q = Discharge
A = Area
d = Depth

For our analysis the slope can be considered constant and the roughness is essentially the same for all subsections between the top bank control. Therefore, Q is essentially proportional to $Ad^{2/3}$ (conveyance).

Varying Discharge - Conveyance Consistency

The above definition shows that for a constant discharge (water and sediment) the conveyance would equalize for all sections. However, discharge is never constant on an alluvial river system and at any instantaneous point in time each section is scouring or filling subject to previous experienced and ongoing hydrographs. Figure 1 (Anding 1970) depicts this continuous scour and fill. This figure plots average sediment discharge against water discharge for several meandering (bendways) and straight (crossing) reaches on the Mississippi River. This figure shows that for a water discharge of 1,000,000 cfs, the average meandering reach transports approximately twice as much sediment as the average straight reach, indicating scouring in the bendway and filling in the crossing. The exact opposite is true for a discharge of 300,000 cfs.

This data has also been confirmed by actual field surveys taken during high water on the Mississippi River which shows bendways scouring as much as 60 feet during the high flow and then refilling to their original elevation as low water approached. Comparable magnitude of fill at crossings during high flow and later scour during the falling river stages have been recorded. This continuous scour/fill relationship is why the measured total conveyance for a given discharge is not equal at all individual sections along any given reach at one point in time. Simply stated the conveyance at each section is attempting to equalize; however, the scouring or filling never has time to adjust to conditions as imposed by the previously experienced and the continually changing hydrographs. Even though this variation occurs at individual sections, studies of dredging, non-dredging, sinuous, and straight reaches over a period of time spanning 24 years of record show a very consistent mean reach conveyance for all reaches over time as shown in Figure 2 (less than 15% deviation).

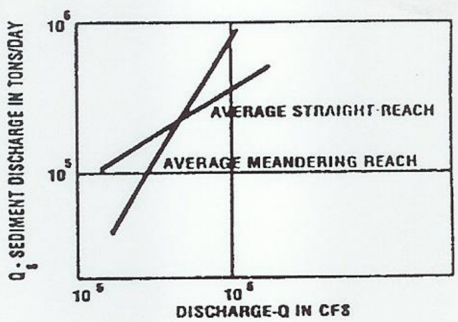

Figure 1
Sediment/Water Discharge

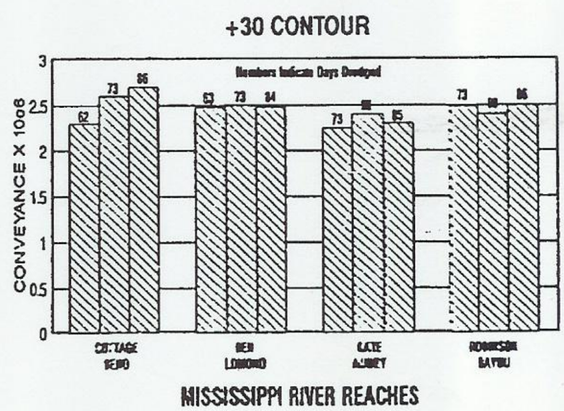

Figure 2
Total Channel Conveyance .

This consistency shows that Q is essentially proportioned to conveyance considering mean reach conveyance as related to mean discharge and supports the concept that if given time to equalize for a given flow, all sections would become proportional. Field observation also shows that the preponderance of the perpetual scouring and filling at individual sections tends to occur in the main channel areas and that the fringe areas of the channel tend to be less dynamic. By analyzing these fringe areas (outside a predetermined main channel width) and recognizing that the mean conveyance for the mean discharge remaining essentially constant, an empirical relationship can be developed to show how much conveyance can be allowed in these areas and still insure that the fill cycle for any given section does not exceed the required elevation in the navigation channel to insure maintenance free navigation.

MR&T DESIGN CURVE

General

The first example of successfully using this conveyance approach in the format shown is on the Channel Improvement Feature of the MR&T project, which provides for stabilizing the Mississippi River from Cairo, Illinois, to the Head of Passes, Louisiana, along a satisfactory alignment to protect the flood control system and provide dependable, economical navigation. One of the major components for construction of this feature are stone dikes which are typically used in crossings and straight reaches to control channel width and alignment to reduce or prevent periodic maintenance dredging. Least cost analysis of dredging problem areas show that it is almost always cost effective to prevent the maintenance dredging with dike construction. The effectiveness of this feature is demonstrated by an analysis of dredging and dike construction date which shows a direct relationship between the length of dike constructed and the nuumber of days dredged per year (see Figure 3). As dike construction increased, dredging decreased proportionately. Historically, most of the work on this feature was to control the overall general channel alignment. However, in recent years as the dike program approached 70% completion, additional refinement in design was needed to more directly address areas which continued to have persistent dredging problems. The best data available to develop this refined design procedure was believed to be the river itself. There were numerous reaches where extensive dredging was required until dikes were constructed and now never required maintenance dredging along with reaches with dikes where dredge persisted. As a result, the subsequent design curve was developed.

Section Control and Channel Layout

The engineer has only three ways of controlling the navigation channel depth without dredging. The first way is through overall alignment of the main channel. Under ideal conditions with the proper alignment, sinuosity adequate depth can be maintained by stabilizing the banklines with revetments and no or only minimal dike construction. However, for obvious reasons, the ideal alignment cannot always be maintained and in these instances, the only other two controls available is the width of the main channel and the height of the dikes used to control this width. By controlling these two parameters, the conveyance outside of the main channel can be limited to the extent required to ensure a 9-foot navigable depth over the highest point in the 300-foot navigation channel as shown on Figure 4. To develop this empirical relationship, a total of 65 cross-sections out of 42 separate reaches were analyzed. Thirty-one of these reaches contained dikes and required no maintenance dredging, while 11 contained dikes but still required dredging.

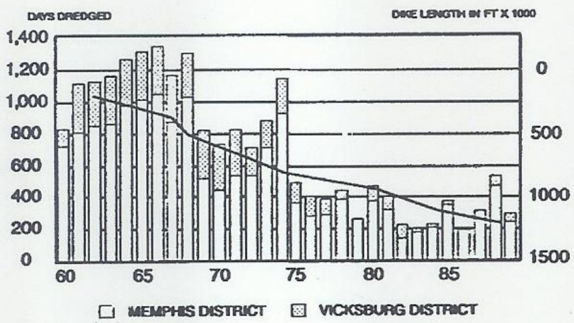

Figure 3
Dredging/Dike Construction

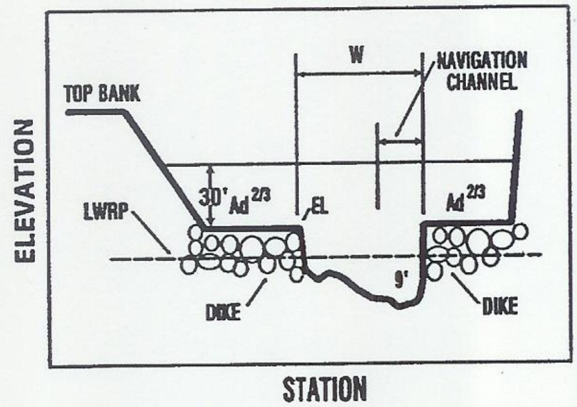

Figure 4
Typical Section

Depth/Conveyance

Using the 65 cross-sections discussed above, the conveyance ($Ad^{2/3}$) outside of the main channel above the top of dikes and below an elevation of LWRP+30' (approximately bankfull stage) was computed. This conveyance was then plotted against the width of the main channel as shown on Figure 5. Sections located where maintenance dredging is required were

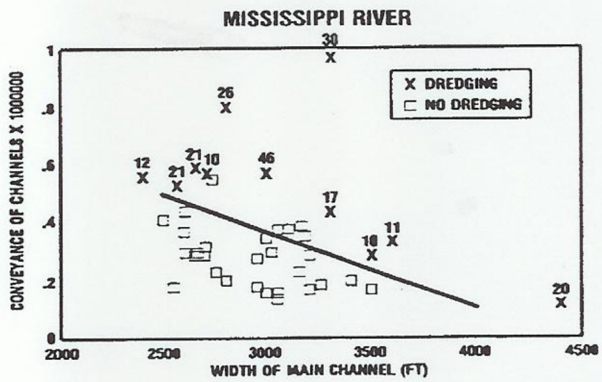

Figure 5
MR&T Design Curve

plotted with X's and those where no maintenance is required are plotted as squares. By comparing the plots, it becomes obvious that two distinct groupings emerge with the areas where maintenance dredging is required having considerably more conveyance outside the main channel for a given main channel width. The line of separation between these data becomes a design line whereby one can determine the conveyance which can be allowed outside the main channel for a given main channel width. For example, for a channel width of 3,000', a conveyance of 380,000 can be allowed outside of the main channel before maintenance dredging would be expected. Using this line, the elevation of the top of stone in the dikes can be determined to appropriately control conveyance.

BATON ROUGE TO GULF DESIGN CURVE

General

The second example of using this conveyance approach is on the Baton Rouge to Gulf project. This project provides a 500' wide with 40' draft channel, which is now being deepened to 45'. The curve developed for the MR&T project is not applicable for designs in the reach; however, the design concepts are still valid. Instead of using width between dikes (no dikes previously constructed in this reach), the width was established and conveyance was plotted against depth. The idea is to determine how the conveyance must be reduced in order to gain specific depth.

Section Control and Channel Layout

The same basic controls are available for this project as were available for the MR&T project. For example, by establishing the main channel width and dike height the conveyance outside of the main channel can be controlled to the extent required to insure navigable depth over the highest point in the 500-foot navigation channels. To develop this empirical relationship between this outside conveyance and available navigation depth 17 crossings were selected and analyzed. Nine of these crossings required no maintenance dredging for the 40 or 45-foot project. On the comprehensive survey of the crossings a 2,000 foot main channel and the 500-foot navigation channel was defined as shown on Figure 6. The 500-foot wide navigation channel is established by law and the 2,000-foot

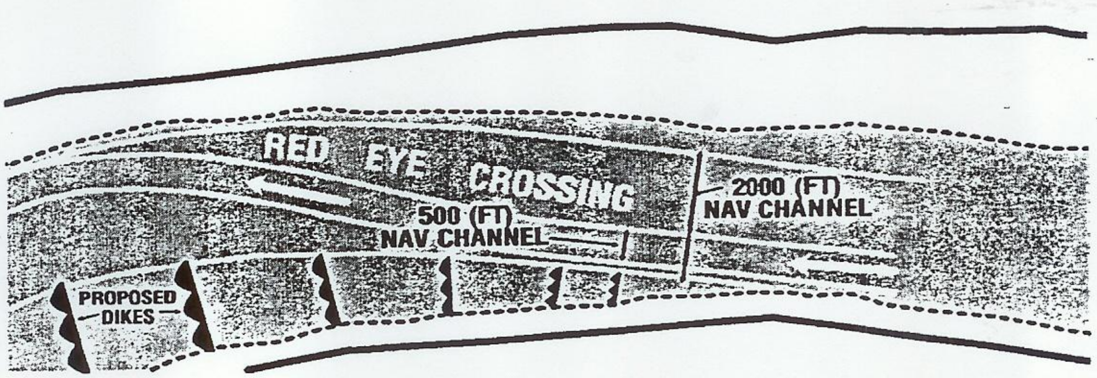

Figure 6
45' Channel Plan

main channel width was selected as being representative of other existing top bank control crossings where no maintenance dredging has been required for the 40-foot project and would not be expected for a 45 to 55 foot project.

Depth/Conveyance

Using the layouts for the 17 crossings discussed above a typical section generally through the highest point for the crossing in the 500-foot navigation channel was selected. From this section the conveyance ($Ad^{2/3}$) outside of the main channel below an elevation of LWRP+25' (approximately topbank) as previously defined was computed. This conveyance was then plotted against the highest point in the crossing taken from the most recent comprehensive survey within the defined 500-foot navigation channel as shown on Figure 7. Sites which have or are expected to have maintenance dredging are shown as solid squares and sites where no maintenance dredging has occurred or is expected to occur are shown as open squares.

A statistical analysis using least squares was then used to develop the heavy line which represents the average data for the 17 points. However, for the purpose of design, average values should not be used since for this condition depth would be unavailable for 50 percent of the locations. Therefore, a band was developed which encompasses a preponderance of the data and the safe side of the bend was selected as the design curve. For example, if a 40-foot deep navigation channel is desired, the conveyance outside of the main channel must be no more than approximately 400,000. Likewise, if a 45-foot deep navigation channel is desired the conveyance outside of the main channel must be no more then

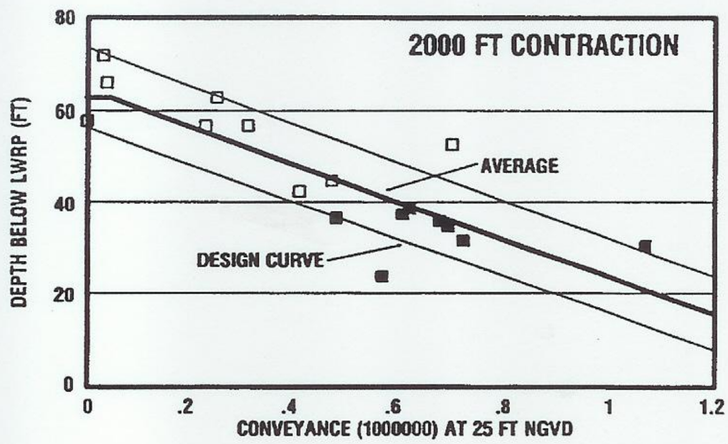

Figure 7
Baton Rouge to Gulf Design Curve

approximately 300,000. As an additional check for consistency of data, it is interesting to note that by using the design curve for a 40 foot required depth all of the sites which require maintenance dredging fell to the right of the line, indicating too much conveyance outside of the main channel. The same analysis using the average data line shows 4 sites on each side of the line, confirming the 50 percent condition previously discussed for the average conditions. With the development of this curve, stone dikes can be located in plan at a problem crossing as shown in Figure 6 and the top of stone elevation computed to obtain the desired conveyance over the dike.

SUMMARY

This paper presents some of the problems associated with developing design formula and criteria for moving large volumes of water and sediment downstream in a manner consistent with project purposes. The importance of analyzing actual river responses which incorporate all variables is stressed. Two empirically developed design curves which are currently being used on the Mississippi River are presented. These curves give the designer a way to mathematically compute dike height and length. As previously noted, these design curves are reach specific; however, the concepts and relationships developed can be used to develop similar design curves for other alluvial river systems.

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