CECW-ED Engineer Manual 1110-2-1611	Department of the Army U.S. Army Corps of Engineers Washington, DC 20314-1000	EM 1110-2-1611 31 December 1980
	Engineering and Design LAYOUT AND DESIGN OF SHALLOW-DRAFT WATERWAYS	
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CECW-ED

27 August 1997

Errata Sheet

No 1

ENGINEERING AND DESIGN

Layout and Design of Shallow-Draft Waterways

EM 1110-2-1611
Change 3

31 July 1997

Signature page: Paragraph 1, change date from 10 Feb 97 to 3 1 Dec 80

	DEPARTMENT OF THE ARMY	EM 1110-2-1611
	U.S. Army Corps of Engineers	Change 3
CECW-ED	**Washington, DC 20314-1000**	

Manual
No. 111O-2-1611 — 31 July 1997

Engineering and Design
LAYOUT AND DESIGN OF SHALLOW-DRAFT WATERWAYS

1. This Change 3 to EM-1110-2-1611, l0 Feb 97:

   a. Updates Chapter 7, Section 5, to provide detailed guidance on the design of stone spur dikes and additional information on other types of river training structures.

   b. Updates Chapter 15 to provide guidance on conducting numerical model studies.

   c. Updates the Table of Contents to reflect the changes in Chapters 7 and 15.

2. Substitute the attached pages as shown below:

Chapter	Remove page	Insert page
Table of Contents	iii thru vii	iii through vi
7	7-3 thru 7-6	7-3 through 7-17
15	15-1 thru 15-4	15-1 through 15-9
Appendix A	A-l	A-l through A-2

3. File this change sheet in front of the publication for reference purposes.

FOR THE COMMANDER:

Otis Williams

OTIS WILLIAMS
Colonel, Corps of Engineers
Chief of Staff

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
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EM 1110-2-1611
Change 2

DAEN-CWE-HD

Engineer Manual
No. 1110-2-1611

29 October 1982

Engineering and Design
LAYOUT AND DESIGN OF SHALLOW-DRAFT WATERWAYS

1. A correction of the quotation from Section 5 of the 1915 Rivers and Harbors Act is indicated with an asterisk at the beginning and end of each change.

2. Substitute the attached pages as shown below:

Remove old pages	Insert new pages
4-l and 4-2	4-l and 4-2

3. File this change sheet in front of the publication for reference purposes.

FOR TEE COMMANDER:

for John J. [illegible] Jr.

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EM 1110-2-1611
Change 1

Engineer Manual
No. 1110-2-1611

15 March 1982

Engineering and Design
LAYOUT AND DESIGN OF SHALLOW-DRAFT WATERWAYS

1. Chapter four of this manual required several additions and revisions and are so indicated with an asterisk at the beginning and end of each change.

2. Substitute the attached pages as shown below:

Remove old pages	Insert new pages
i and ii	i and ii
4-l thru 4-15	4-l thru 4-17

3. File this change sheet in front of the publication for reference purposes.

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fr JAMES W. RAY
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EM 1110-2-1611

Engineer Manual
No. 1110-2-1611

31 December 1980

Engineering and Design
LAYOUT AND DESIGN OF SHALLOW-DRAFT WATERWAYS

1. Purpose. The purpose of this manual is to provide guidance for planning, layout and design of shallow-draft waterways.

2. Applicability. This manual applies to all field operating activities having responsibility for the design of civil works projects.

3. General. Development of waterways for navigation involves channel excavation, rectification, bank stabilization, training structures, modification and/or construction of bridges, and in some cases, the construction of locks and dams. This manual covers some of the principal factors that should be considered in the design and solutions that have been successful in avoiding or eliminating undesirable conditions. Unless these factors are applied, adverse conditions or delays could occur to such an extent that traffic potential of the waterway would not be fully developed. It is important that conditions resulting from these works be satisfactory and adequate for the traffic anticipated and provide a high degree of reliability.

FOR THE CHIEF OF ENGINEERS:

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EM 1110-2-1611

Engineer Manual
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31 December 1980

Engineering and Design
LAYOUT AND DESIGN OF SHALLOW-DRAFT WATERWAYS

Table of Contents

CHAPTER 1

INTRODUCTION

1-1. Purpose. This manual provides guidance in the planning, layout, and design of shallow-draft waterways, factors that should be considered and solutions that have been successful in avoiding or eliminating undesirable conditions.

1-2. Applicability. This manual applies to all field operating activities having responsibilities for the design of civil works projects.

1-3. References and Bibliography. See Appendix A.

1-4. Background. Development or improvement of waterways for shallow-draft navigation involves the solution of many problems, particularly when the use of natural streams is involved. These problems are concerned with the factors that could adversely affect the safe and efficient movement of traffic, water quality, and/or environment. Unless these factors are considered and incorporated in the design of the project, hazardous conditions or delays could occur to such an extent that commercial traffic would not be econimically competitive with other modes of transportation or the traffic potential of the waterway would not be fully developed. Development or improvement of waterways for navigation usually involves large expenditures for channel excavation, rectification, and stabilization; training structures; modification and construction of bridges; and in many cases, the construction of locks and dams. Since the modifications and structures are provided primarily for navigation, it is important that conditions resulting from these works be satisfactory and adequate for the traffic anticipated and provide a high degree of reliability.

1-5. Scope. This manual covers some of the principal factors that should be considered in the design and improvement of inland waterways for commercial traffic consisting mostly of barge traffic rather than for seagoing vessels or freighters using the Great Lakes. Some of the factors affecting the safety and efficiency of waterways that are discussed include: types of waterways; environmental considerations; equipment in general use on connecting waterways; alignment and velocity of currents; channel alignment and dimensions; number, location, and size of locks; harbors and docking facilities available; visibility; bridge location and clearances; ice and debris; and weather conditions.

CHAPTER 2

PRELIMINARY PLANNING AND LAYOUT

2-1. Justification. Justification for the development of a waterway for navigation is based on feasibility studies covering the amount and type of traffic that could be developed, commodities that would be moved on the waterway, effect on the environment and economic development of the region, and estimated cost of construction, maintenance, and operation. This should include a study of the region, centers of population, resources that would be developed, characteristics, potentials and history of the region, and cost of moving commodities by water compared with other modes of transportation.

2-2. Preliminary Planning. Initial planning would require the collection and evaluation of all pertinent data including special surveys needed to evaluate the probable short- and long-term effects on local environment and development of the waterway. The information should include topographic and hydrographic data, hydrologic and hydraulic data, geological information, soil characteristics and location of existing and proposed highways, railroads, bridges, and industrial complexes. This information would be required to determine routes to be followed, type of waterway that could be developed most economically, and estimated cost.

2-3. Evaluation of Existing Streams. The first step is to evaluate the existing river systems to determine their ability to accommodate navigation. The necessary studies are channel widths and depths at various seasons, sediment load, extent of bank erosion, flood magnitude and frequency, and environmental factors such as water quality and biologically important habitats.

2-4. Commodities to be Moved. The next step in designing a commercial waterway is to develop an estimate of the expected commodity shipments. These shipments will establish the requirements to be accommodated. Shipments can be broken down to commodities, season in which moved, return trip traffic, and needed barge type and size. Also needed are the trip time and tow sizes necessary to make the waterway route more economical than other modes of transportation.

2-5. Features Considered. This background information can now be applied to design of a waterway. The design procedure requires optimization of the following interrelated features:

a. Open river or canalization.

b. Channel size.

c. Tow size.

d. If canalized, size and number of locks.

2-6. Waterway Types. The type or types of waterways that could be developed will vary with local conditions. The types normally considered are open river, canalized streams with locks and dam, land-cut canals, or a combination of one or more of these types. Each type has its advantages and disadvantages which have to be considered.

2-7. Open River. The towing industry would prefer open-river navigation since it would eliminate delays normally encountered in passing through locks, but this is not practical on many streams because of their characteristics and local restraints. Many problems are associated with open-river navigation, and development and maintenance of this type of waterway usually involve some channel rectification, training and stabilization structures, maintenance dredging, and navigation aids. Changes in river stage and discharge produce changes in channel width and depth and in some cases channel alignment. The first cost of developing this type of waterway is generally less than that with other types but requires continuous surveillance and marking of the channel and considerable maintenance. Open-river navigation is maintained on the Mississippi River below St. Louis, the Missouri River, and the Columbia River below Bonneville Dam.

2-8. Canalized Streams. Canalized streams involve the construction of locks and dams to maintain adequate depths for navigation during periods of medium and low flows. Locks and dams would be required in streams having steep gradients with velocities too high for navigation or where conditions make it impractical to develop the required depths naturally because of rock outcrops, sediment movement, and other factors that could adversely affect navigation and flood-carrying capacity of the stream. Even with locks and dams, some channel improvement and regulating and stabilization structures and channel maintenance will be required. The principal disadvantage of this type is high initial cost and delays caused by tows passing through each lock. Canalized waterways usually have lower velocities and greater channel width and depth through most of the reach of the pool during controlled riverflows. Examples of canalized waterways are the Ohio and Monongahela Rivers, Mississippi River above St. Louis, Mo., and the Arkansas River. Locks might also be required in channels through estuaries, bays, near the mouths of some streams, and in some sea-level canals to prevent salt-water intrusion or minimize the effects of tides and differences in water levels with connecting waterways.

2-9. Canals. Land-cut canals have been used to connect two bodies of water, to bypass rock outcrops and rapids, and to reduce the length or curvature of the navigable channel. Canals can parallel existing streams or continue overland to reach specific destinations. Construction of canals can be expensive depending on the amount and type of excavation, land acquisition, and availablity of disposal areas. When connected to an existing stream or other body of water, locks might be required in the canal. In order to reduce the amount of excavation, canals might be routed through shallow lakes and estuaries. Stabilization structures might be required along the banks of the canal to reduce erosion of the banks due to waves created by traffic and wind. Canals tend to be narrow and shallow to minimize cost and could be affected by surges resulting from lock filling or emptying when relatively high-lift locks are used. Examples of land-cut canals are the Chain of Rocks Canal near St. Louis, Mo., New York State Barge Canal, and the intra-coastal canals.

2-10. Basis of Selection. Selection of the type of waterway adopted will depend on the amount and type of traffic that would be developed; characteristic of the equipment in general use; channel alignment and dimensions required; sedimentation problems to be resolved; safety, efficiency, and dependability; environmental effects; and comparative cost of construction, operation, and maintenance.

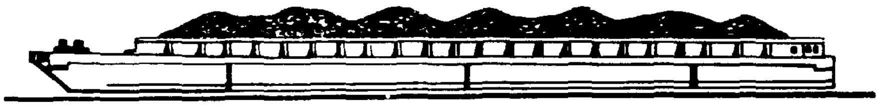
2-11. Cost Estimates. A series of layouts with cost estimates are needed to develop optimized costs. These life cycle cost estimates should include initial construction cost, maintenance cost, and replacement cost. Each of the layouts is required to move the required tonnage but each will have a different trip time. This trip time is translated into benefits. The comparison of project costs versus benefits will provide the basis for selection of the optimum layout. Generally, fewer locks are cheaper than a greater number of lower-lift locks. Economy should consider both first cost and maintenance and operation cost without sacrifice of safety, efficiency, and dependability.

CHAPTER 3

WATERWAY TRAFFIC

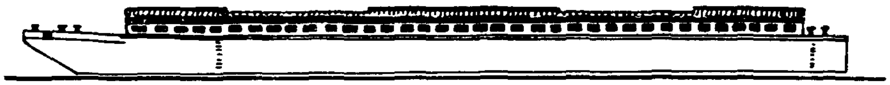
3-1. General. The development of inland waterways for navigation usually can only be justified on the basis of commercial traffic. Therefore, design of the waterway should consider the types of equipment that would be using the waterway and their principal characteristics. River commerce in the United States is handled chiefly by barge tows consisting of a towboat pushing one or more barges, depending on the characteristics of the waterway, facilities provided, type of cargo, and size and power of the towboat. The tow speed and direction are controlled by the towboat, which is at the stern; the head of the tow is at the other end, sometimes from one barge length to more than 1200 feet away. Towboats vary in size, power, and maneuverability and, therefore, in their capability for handling loads under various conditions. Figure 3-1 indicates some of the equipment in more general use in the United States at this time (1979) and figure 3-2 indicates some of the barge and tow configurations used by the towing industry.

3-2. Towboat Controls. The towboat pilot is usually a considerable distance from the head of the tow, and his only means of control of the tow(s) is the action of the towboat rudder and propeller screws. The pilot's control of the tow depends on the maneuverability and power of the towboat, and the ability to anticipate the effects of currents, navigation aids provided, and visibility. The power of the towboat and the action of the rudder affect the movement of the tow, as do the direction and velocity of currents, wind, ice, drift, and channel dimensions. The towboat rudder or rudders develop a side thrust when placed at an angle to the direction of flow. This thrust is proportional to the area of the rudder affecting the currents, the angle of the rudder to the currents, and the square of the velocity of the currents directed against the rudder by the propeller in relation to the speed of the towboat. When a towboat is reducing speed in relation to the velocity of currents, it is losing rudder power; and when moving in the same direction and at the speed of the currents, the tow has no rudder control. When a towboat changes directions, the action of the rudder moves the stern of a forward-moving tow in a direction opposite to that of the turn. The pivot point of the turn from a standing position in slack water is some distance forward of the midpoint (about 30 percent of its length from the head of the tow). When the tow is under way and proceeding ahead, the pivot point moves forward and could be some distance beyond the head of the tow depending on the speed of the tow and the direction of the currents in relation to that of the tow. This explains why the stern of a tow will not necessarily follow the same path as the head when



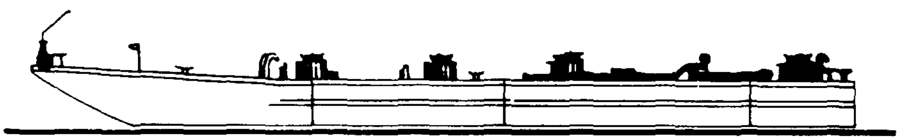
OPEN HOPPER BARGES

TYPE	LENGTH FEET	BREADTH FEET	DRAFT FEET	CAPACITY TONS
STANDARD	175	26	9	1000
JUMBO	195	35	9	1500
SUPER JUMBO	250-290	40-52	9	2500-3000



COVERED HOPPER BARGES

TYPE	LENGTH FEET	BREADTH FEET	DRAFT FEET	CAPACITY TONS
STANDARD	175	26	9	1000
JUMBO	195	35	9	1500



INTEGRATED CHEMICAL AND PETROLEUM BARGES

LENGTH FEET	BREADTH FEET	DRAFT FEET	CAPACITY TONS
150-300	50-54	9	1900-3000

TOWBOATS

LENGTH FEET	BREADTH FEET	DRAFT FEET	HORSEPOWER
65-160	24-50	5-9	300-7000

Figure 3-1. Predominant barge and tow types

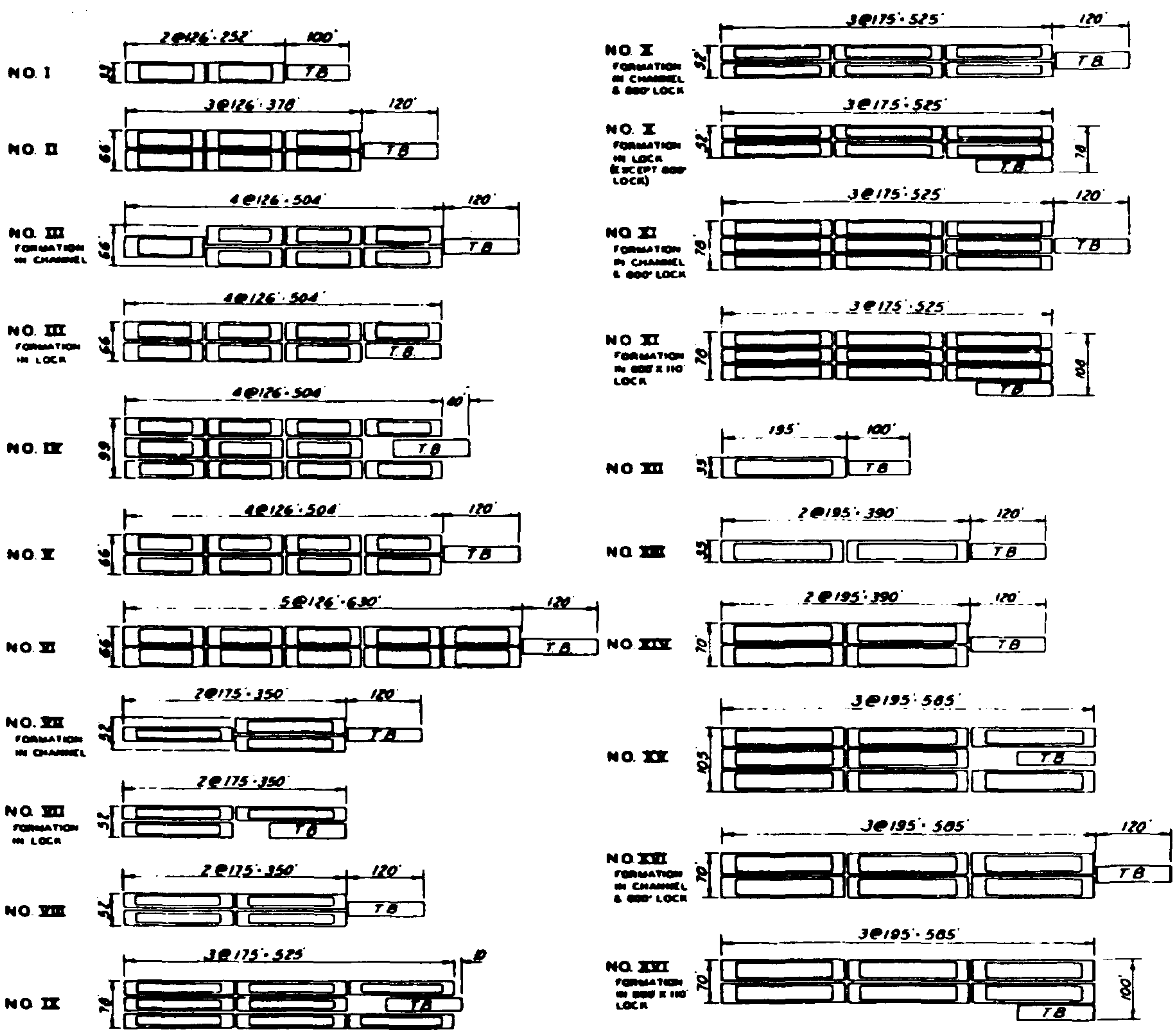


Figure 3-2. Typical arrangements for barge tows

turning, going around bends, or attempting to compensate for adverse currents.

3-3. Maneuverability of Tows. The maneuverability of towboats varies depending on the size and number of rudders, power versus load of the towboat, and special equipment, such as Kort nozzles or bow steerers. Most towboats are equipped with flanking rudders which operate when the screws are reversed (backing) and which can be used in negotiating sharp bends and to assist in maneuvering the tow for the approach to the lock or in negotiating critical reaches. A flanking maneuver consists of reversing the screws to retard the movement of the tow and thus permitting currents to swing the head of the tow in the desired direction or to move the stern of the tow laterally. Since flanking is greatly dependent on the direction and alignment of the currents, the head of the tow cannot always be moved into the desired position by flanking alone. Towboats having independently controlled twin screws can develop a twisting action to provide some control of the movement of the head of the tow by having one screw pushing ahead and the other in reverse. Flanking or maneuvering increases the time and power required to move the load, thereby increasing the cost of operating the tow. Bow steerers, which are power units located in the bow of the towboat or lead barge, can improve the maneuverability of tows considerably; however, for various reasons, these are not in general use. Design of navigation facilities should consider that special steering devices generally will not be available and that some towboats will be operating with power insufficient for the safe handling of their loads.

3-4. Visibility. Good visibility is required to locate channel markers, traffic in the area, bridge piers, and navigation aids. Visibility can be affected by weather conditions such as heavy fog, rain, or snow; channel alignment; and location of the pilot with respect to the head of the tow. Sight distance can be limited by bends in the alignment of the channel, location of islands on high sandbars, or structures along the banks. The effect of tow configuration and elevation of the pilot house is illustrated in figure 3-3. The types of navigation aids available to assist pilots in negotiating critical reaches and the traffic control that could be provided for safety should be determined in coordination with the U. S. Coast Guard during early stages of planning.

3-5. Effects of Currents. Adverse currents can cause or contribute to accidents and delays in navigation. Tows are affected by the velocity and alignment of currents relative to the path of the tow. Currents moving at an angle to the path of the tow are referred to as crosscurrents. These currents can be encountered in river crossings, in bends, near side or divided channels, in the entrances to canals, and in the approaches to locks. The velocity of currents in the stream can affect the intensity of the crosscurrents, increase the time of travel and power required for tows moving in an upstream direction, and affect the maneuverability and control of tows moving in a downstream direction. Wind blowing across the path of a tow, particularly one with empty barges, can also have a serious effect on the maneuverability of the tow.

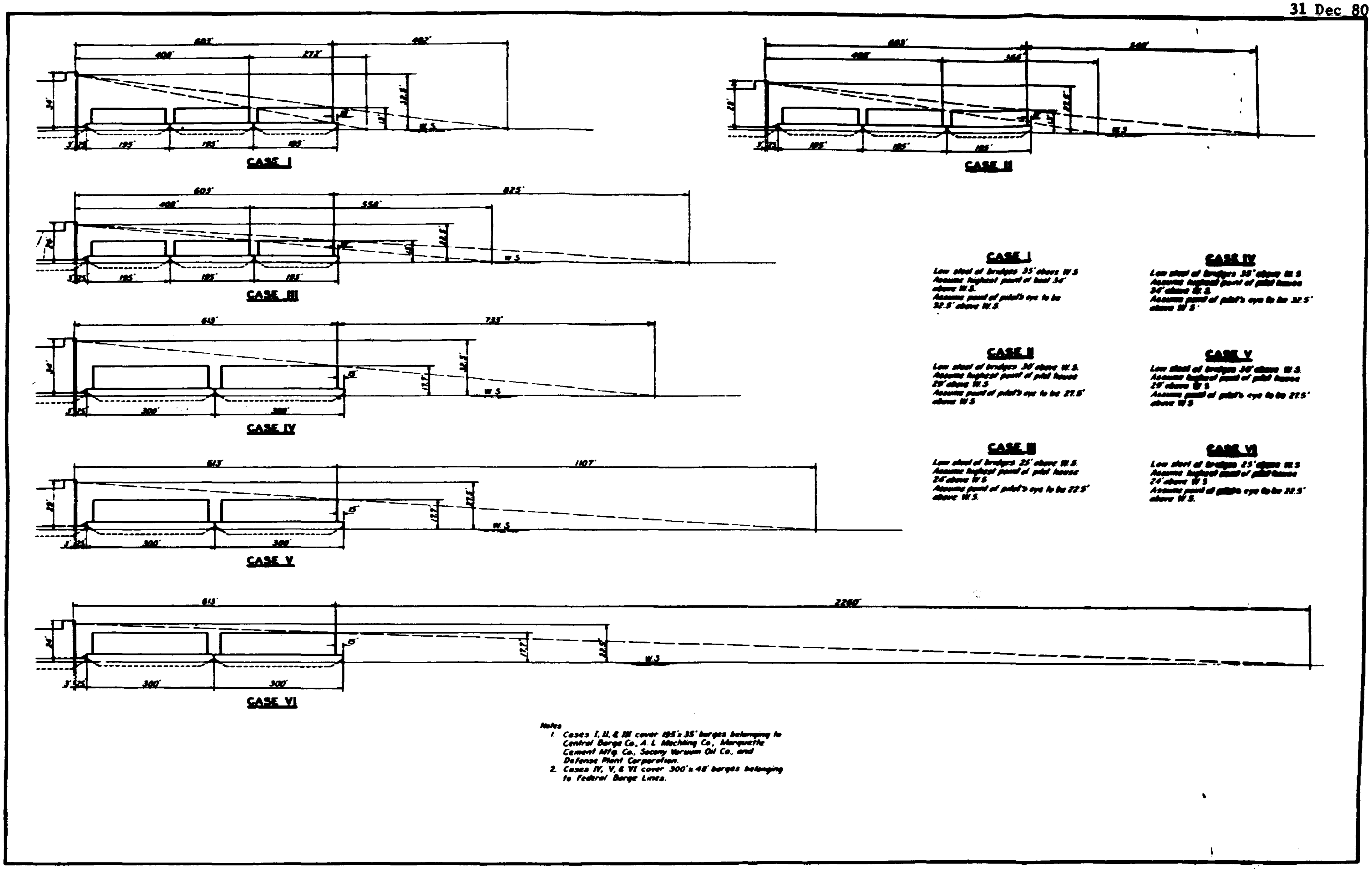


Figure 3-3. Sight analysis

CHAPTER 4

CHANNEL SIZE AND ALIGNMENT

Section I. General

4-1. Channel Characteristics. The type and amount of traffic that can be accommodated by a waterway will depend to a considerable extent on the available width, depth, and alignment of the channel in addition to other factors. Most shallow-draft waterways utilize most or part of an existing stream which consists of alternate bends and straight reaches or crossings between bends. Even canals have bends as required to take advantage of existing lakes or low areas and/or bypassing existing structures or highly developed areas. Because of the characteristics of the equipment using the waterways, the channel dimensions required will vary depending on the alignment of the channel.

* 4-2. Channel Dimensions. Section 5 of the Rivers and Harbors Appropriation Act approved 4 March 1915 outlines the basis for channel dimensions as follows:

> "That in the preparation of projects under this and subsequent river and harbor Acts, unless otherwise expressed, the channel depths referred to shall be understood to signify the depth at mean low water in tidal waters tributary to the Atlantic and Gulf coasts and at mean lower low water in tidal waters tributary to the Pacific coast and the mean depth for a continuous period of fifteen days of the lowest water in the navigation season of any year in rivers and nontidal channels, and the channel dimensions specified shall be understood to admit of such increase at the entrances, bends, sidings, and turning places as may be necessary to allow of the free movement of boats."

Note the statement "unless otherwise expressed" means, specific project authorization takes precedence over the Section 5 general authorization. Where rivers have been canalized, the channel depth referred to will usually be the depth at normal pool. When a width of channel is specified it will be understood to mean the bottom width at project depth in straight segments. Widening on bends is generally needed to allow safe vessel transit. *

4-3. Channel Requirements. Channel dimensions required for navigation will depend on channel alignment, size of tow, and whether one-way or two-way traffic is to be accommodated. One-way traffic can be justified on segments of some waterways when there is a low volume of traffic, passing lanes are provided on long reaches, and close traffic control and communication are maintained. Providing for two-way unrestricted traffic would result in a much safer and more efficient waterway. Channel dimensions and alignment provided can affect construction and maintenance cost and the development of traffic on the waterway. Providing a straight channel can reduce the length of the waterway and require less channel width than with a sinuous channel. However, channels in natural streams tend to meander with most of the length of the low-water channel occurring in bends of various curvature. In streams carrying sediment, long straight reaches would tend to be unstable and difficult to maintain.

Section II. Channel Design

4-4. Channel Cross Section. In determining the channel size, some of the basic criteria used are the sectional area ratio, draft-depth ratio, and maneuverability requirements. Tests have indicated that the resistance to tow movement in a restricted channel decreases rapidly as the sectional area ratio (ratio of the channel area to the submerged tow area) is increased to a value of 6 or 7 and then decreases less rapidly as the ratio is further increased. Resistance to tow movement and power required to move the tow are increased if the draft is more than about 75 percent of the available depth, particularly if the channel has restricted width, such as a canal or a lock.

Section III. Channel in Straight Reaches

4-5. Minimum Width. The minimum channel widths required for safe navigation in straight reaches depend on the type and size of equipment in general use on the waterway, alignment and velocity of currents, intensity of the prevailing wind, how well the channel limits are defined, navigation aids provided, and whether one-way or two-way traffic is permitted. The minimum channel width should provide for the width occupied by the tow, clearance between the tow and channel limits, and clearance between tows for two-way traffic. Operating experience has indicated that the minimum clearance required for reasonably safe navigation in straight reaches should be at least 20 feet between tow and channel limits for two-way traffic, 40 feet for one-way traffic, and at least 50 feet between tows when passing. When structures or mooring areas that could constitute a hazard are located along the channel limit line, greater clearances should be provided. Also, additional clearance should be provided in channels with restricted cross-sectional area or where adverse currents would be encountered. Because of the larger cross section of channels designed for two-way traffic and passing occurs in a relatively short reach, the clearance required between tows and channel limit is less than for channels designed for one-way traffic. As a guide the minimum channel widths required for tows of various sizes are as follows:

Tow Width, Feet	Channel Width, Feet: Two-Way Traffic	Channel Width, Feet: One-Way Traffic
105	300	185
70	230	150
50	190	130*

* Channel widths of less than 130 feet are not recommended for commercial traffic.

* 4-6. Minimum Crossing Distance. Crossings are straight reaches between alternate bends and are common in meandering streams. Tows leaving one bend, usually from along the concave bank, have to cross toward the opposite bank to approach the concave bank of the next alternate bend. The distance required for a downbound tow to make the crossing without flanking or excessive maneuvering will depend on the size of tow, the width of channel, and the alignment and velocity of currents. The average length of channel required for various sized tows to cross from one side of the channel to the opposite side under varying conditions as determined from the results of model studies is shown in Figure 4-1. These results are based on moving minimum-powered tows located adjacent and parallel to one side of the channel crossing to the opposite side with currents generally parallel to the bank lines. In most crossings, currents will tend to move from along the concave bank of one bend toward the concave bank of the next bend downstream, particularly during the lower flows. In such cases, tows can make the crossing in a shorter distance than indicated. Also, tows with greater power and controllability or tows flanking can make the crossing in a somewhat shorter length of channel than that indicated by the results of the model study. Although not exact, the information in Figure 4-1 provides a good general indication of the length of straight reach that should be provided between alternate bends, particularly when short radius bends and limited channel widths are involved. *

Section IV. Channel Widths in Bends

4-7. Orientation of Tows in Bends. The need for additional widths in bends has been known and recognized but little or no information has been available on the amount required. Recent model studies have provided a basis for computing channel widths occupied by tows of various sizes navigating in bends of different curvature. In making a turn, the stern of a tow is moved laterally in a direction opposite to the direction of the turn (figs. 4-2 and 4-3). In negotiating a bend, the tow *

*

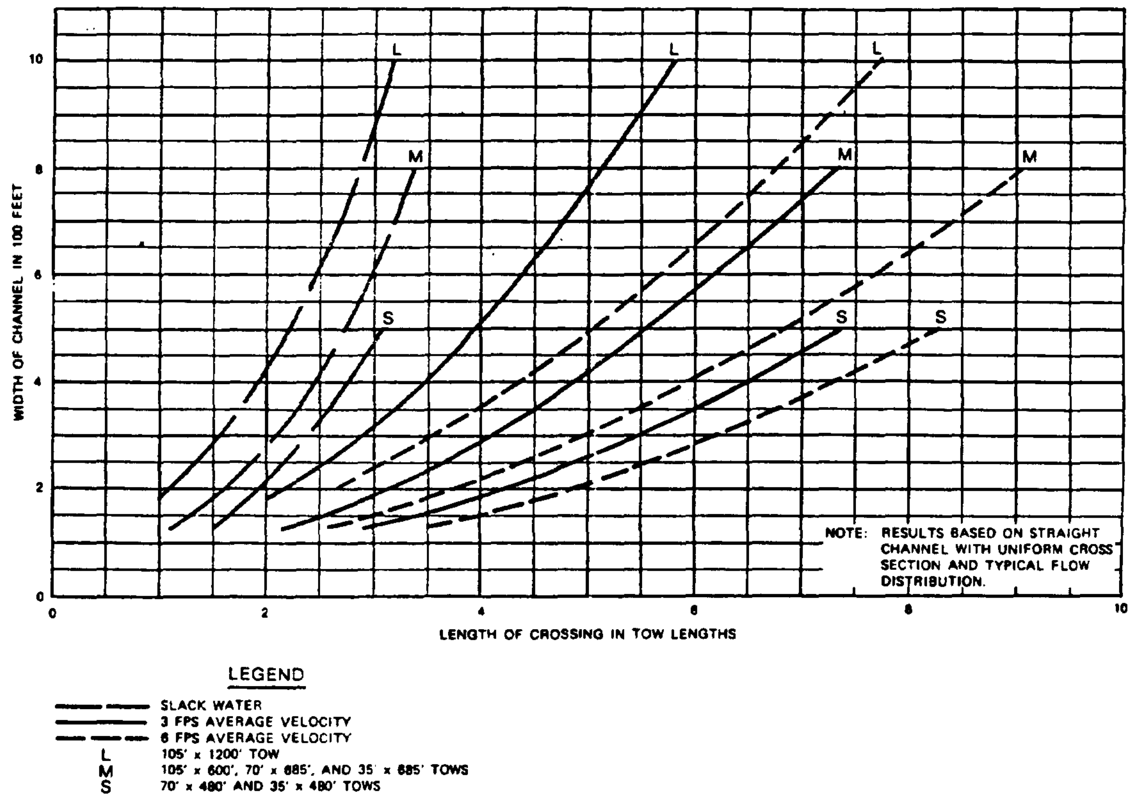


Figure 4-1. Length of channel required between alternate bends *

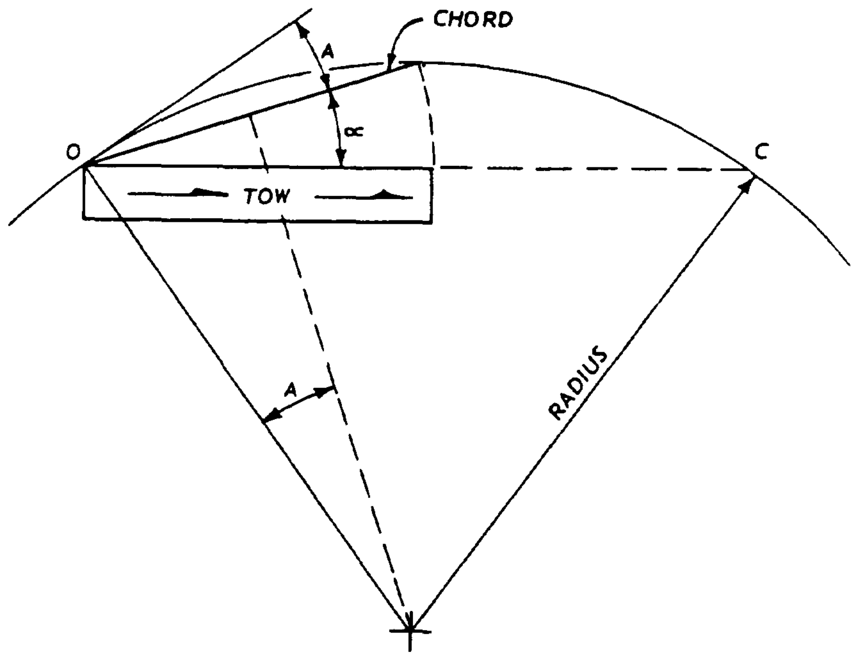
assumes and maintains an angle to the channel alignment which is referred to as the deflection angle (also referred to as drift angle). The width of the channel required is a direct function of the deflection angle assumed by the tow and the length and width of the tow (fig. 4-4). The deflection angle assumed by a tow is dependent on many factors, the most important of which are radius of bend, size of tow, and the length of the bend up to about 90 degrees. Other factors affecting the deflection angle include current alignment and velocity, speed of tow with respect to that of the currents, draft of the tow with respect to channel depth, direction of travel (upstream or downstream), tow driving or flanking when downbound, and alignment and position of tow entering the bend. As a general rule, the deflection angle assumed by a tow increases rapidly as the radius of the bend decreases to less than about four or five times the length of the tow.

4-8. Determining Channel Widths Required in Bends. If the deflection angle assumed by a tow is known, a reasonably accurate channel width

*Figure 4-2. Mosaic showing progressive location and orientation of a downbound 70-foot by 515-foot tow negotiating an actual river bend with low-velocity currents

DOWNSTREAM AND UPSTREAM NAVIGATION

* Figure 4-3. Model reproducing a bend of uniform curvature and radius of 3500 feet with average velocity of 3.0 feet per second. Multiexposure photograph shows progressive movement of each of two 105-foot by 1200-foot model tows with two-way traffic



LEGEND

CHORD = LENGTH OF TOW
A = CHORD ANGLE
α = DEFLECTION ANGLE
O - C = CHORD BASED ON TOW ALIGNMENT MOVING THROUGH THE BENDWAY

* Figure 4-4. Description of deflection angle α

required can be determined from one of the following two equations:

a. $CW_1 = (\sin \alpha_d \times L_1) + W_1 + 2C$

b. $CW_2 = (\sin \alpha_u \times L_1) + W_1 + (\sin \alpha_d \times L_2) + W_2 + 2C + C_t$

where

CW_1 = channel width required for one-way traffic, feet

CW_2 = channel width required for two-way traffic, feet

α_d = maximum deflection angle of a downbound tow, degrees

α_u = maximum deflection angle of an upbound tow, degrees

L = length of tow, feet

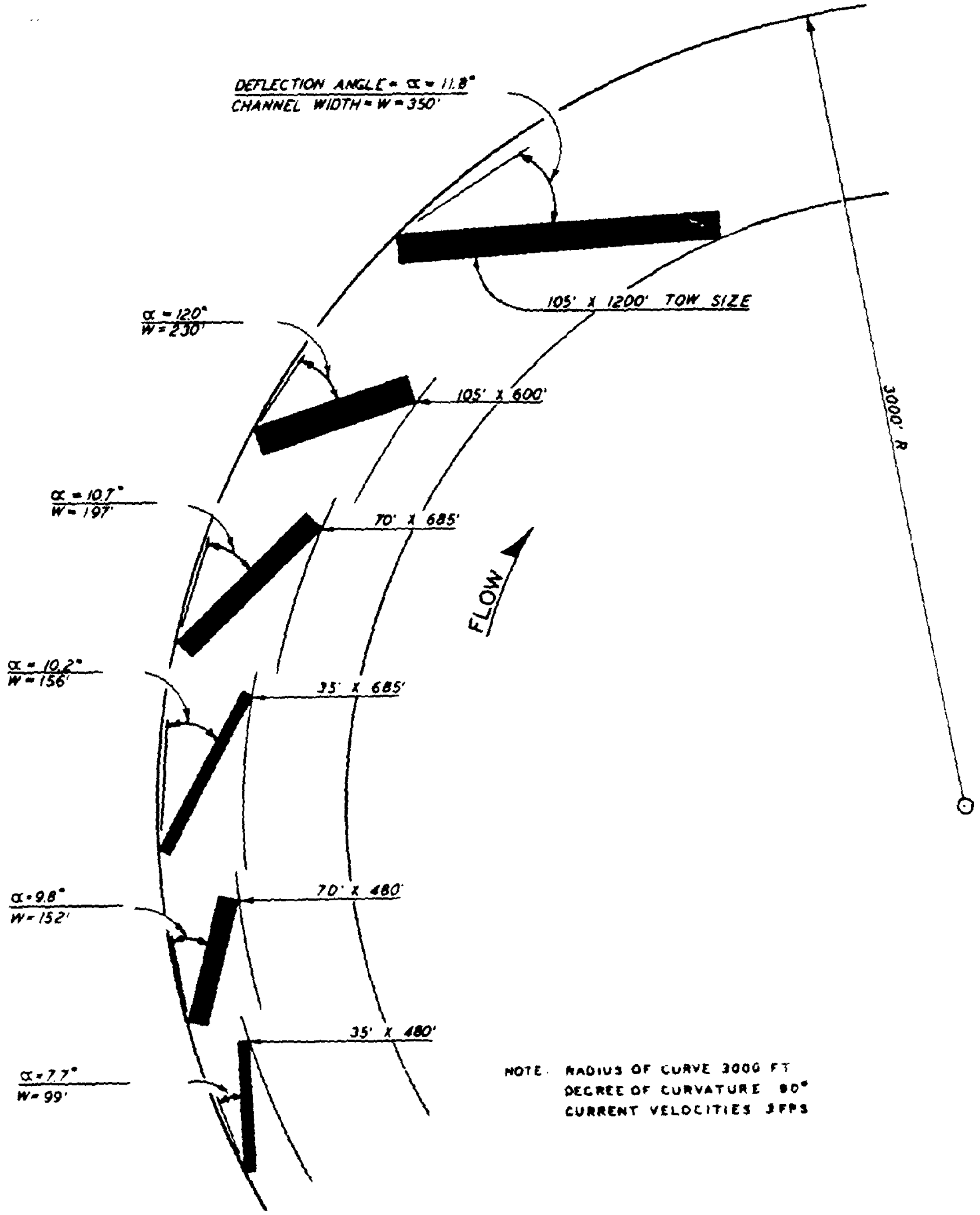
W = width of tow, feet

C = clearance required between tow and channel limit for safe navigation, feet

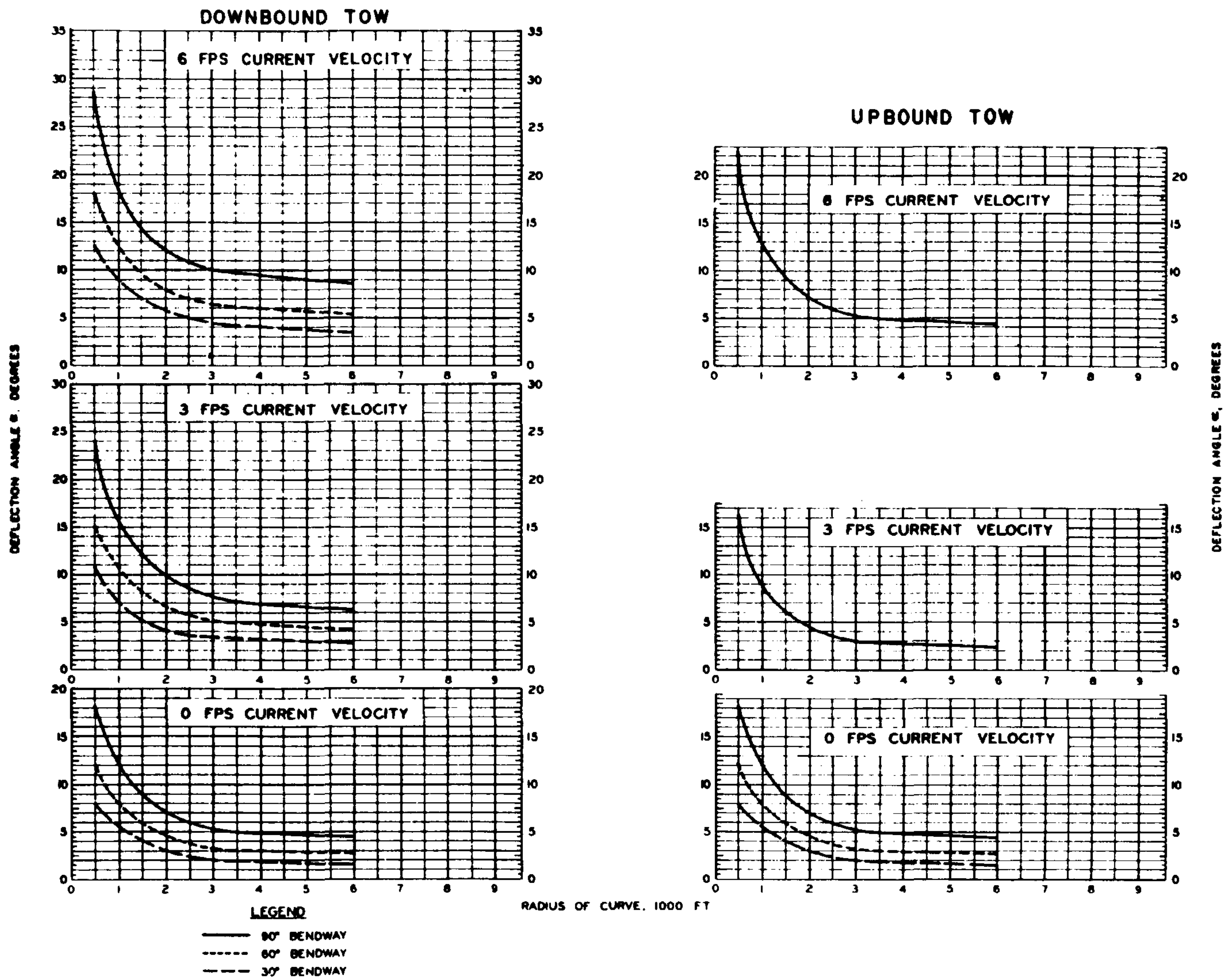
C_t = minimum clearance required between passing tows for safe two-way navigation, feet

* 4-9. Deflection Angles. The deflection angle assumed by different size tows in bends of various curvature based on results of model studies are shown in figures 4-6 to 4-11. These results are based on tests with tows having the minimum power required to navigate the waterway under the conditions indicated. Results for downbound tows are based on driving the bend. Channel dimensions for downbound tows flanking can be approximated by using the deflection angle for an upbound tow. Tows with greater power for the load can develop more rudder control and would assume a smaller deflection angle and require less channel width. Also, tows with greater maneuverability because of independent operation of their screws, specially designed rudders, or auxiliary steering devices will require less channel width than would be indicated by the results of the model tests.

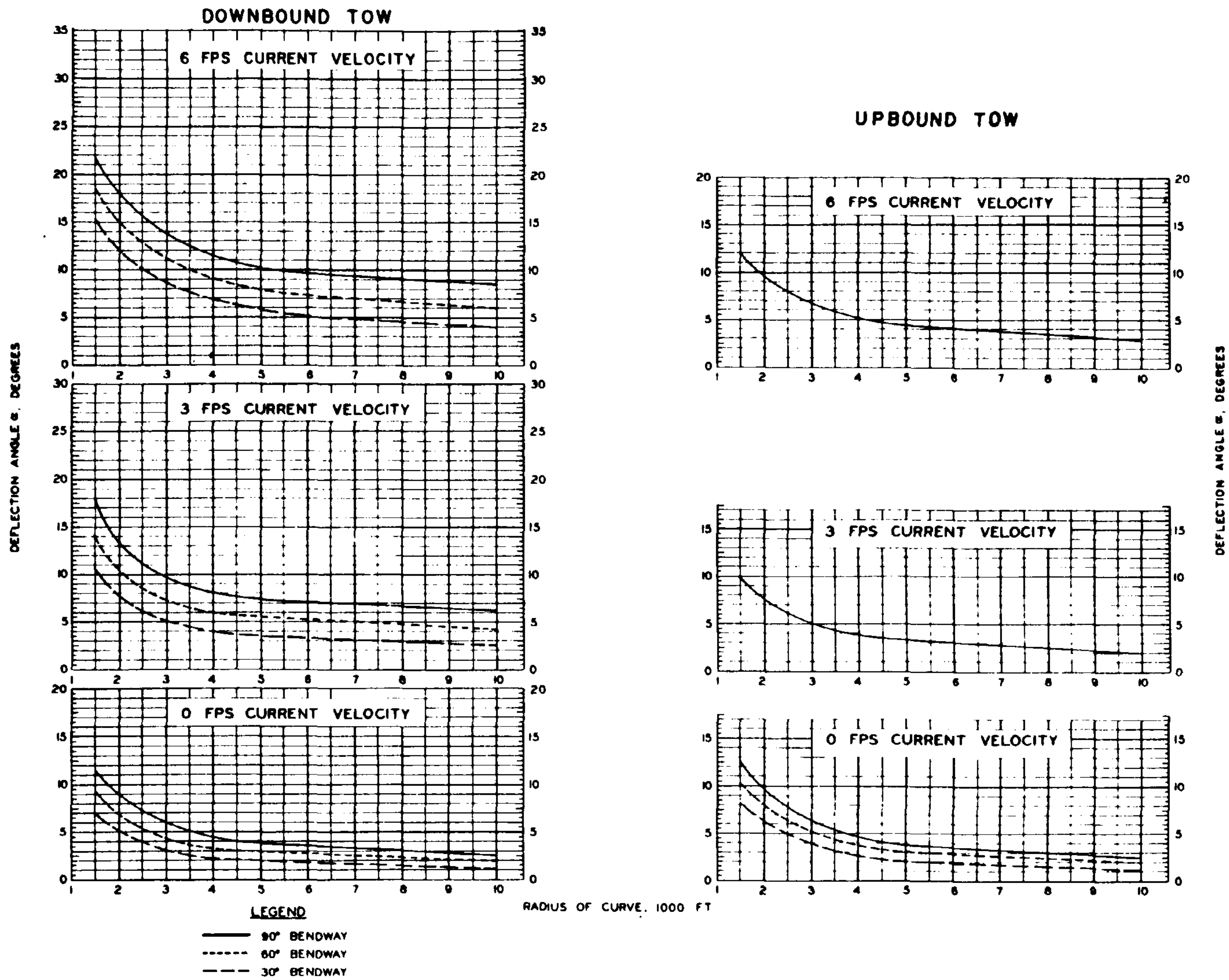
* 4-10. Irregular Bank Line. The preceding information on navigation in bends is based on bends having continuous concave banks. In natural streams having erodible beds and banks, the alignment of the concave banks might include scallops or landward indentations caused by erosion or local bank failures. These irregularities could have an adverse effect on tows navigating the bends and should be considered in the design of the projects. Model studies have indicated that small irregularities in the bank lines would have little or no effect on navigation but that longer irregularities in the downstream one-third of bends would generally have a significant adverse effect. These studies indicate that scallops in the bank line could be hazardous, particularly for downbound tows, when the length of a scallop along the bank is a minimum of one tow length and extends into the bank at least the width of the tow at a depth of 75 percent of the draft of the tow. This type of hazard is caused by currents moving into the scalloped area having a tendency to ground downbound tows moving close along the concave bank or causing them to hit the bank near the lower end of the scallop. In reaches *



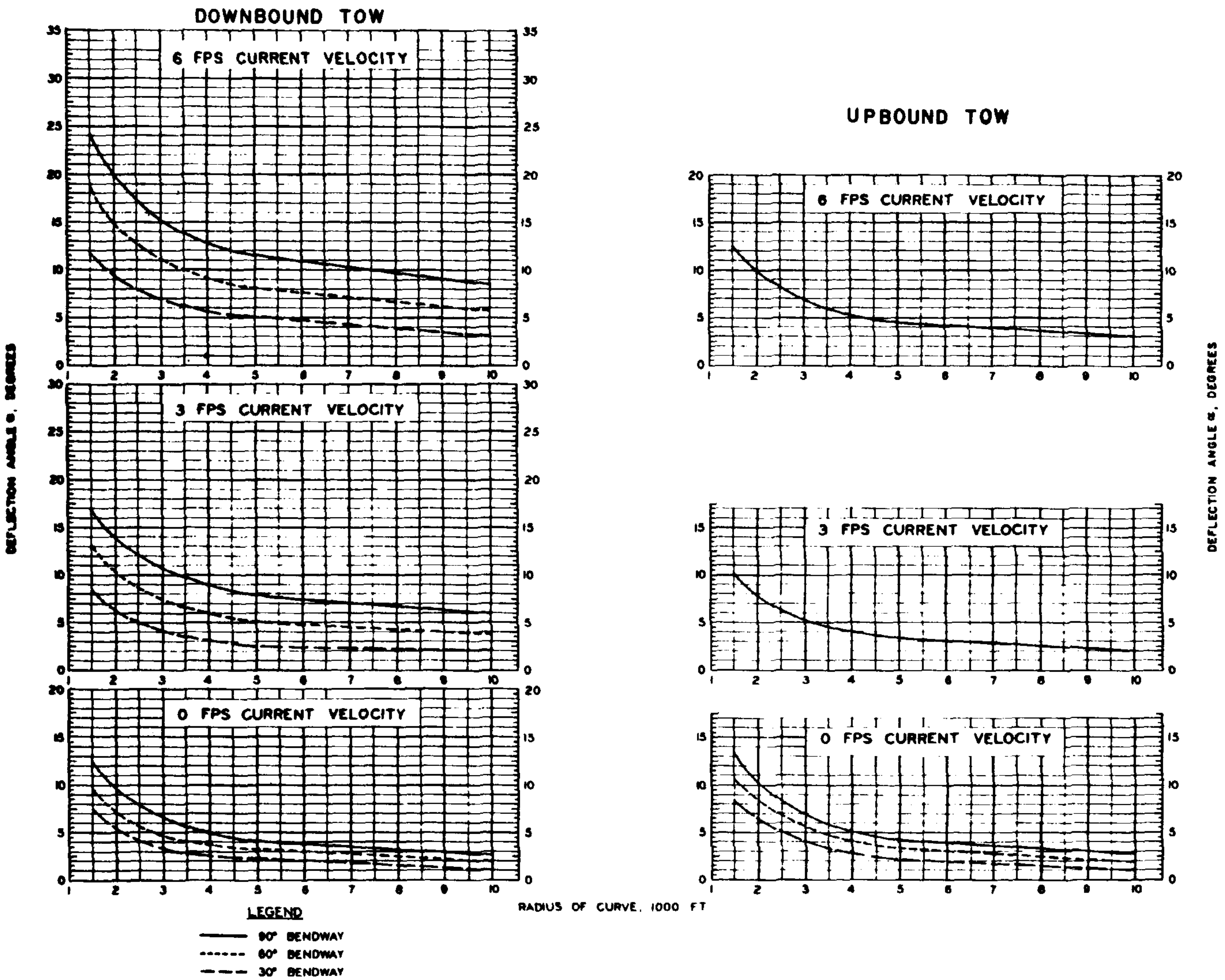
* Figure 4-5. Variation in deflection angle and channel widths occupied by tows of different sizes



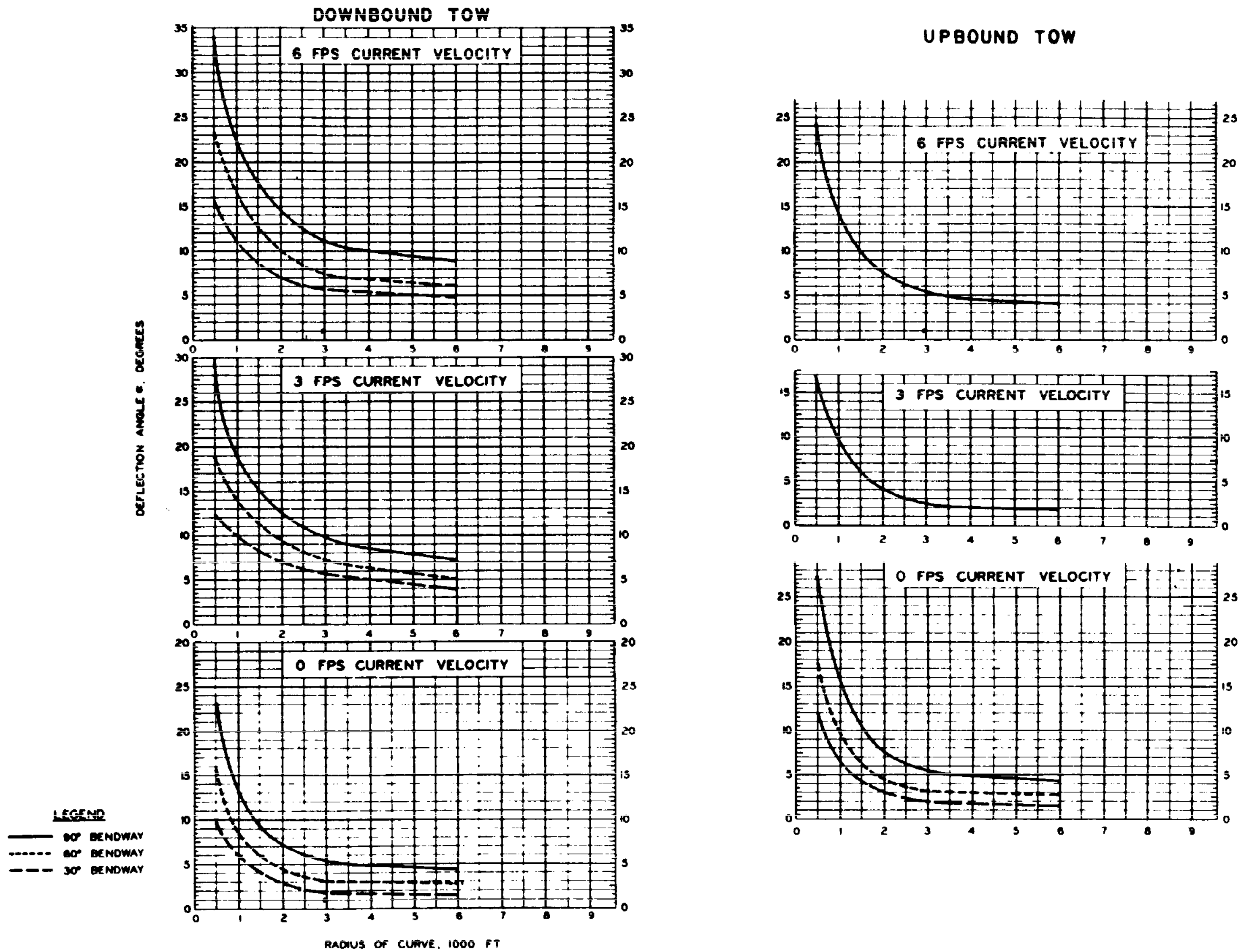
* Figure 4-6. Deflection angle for tows driving through bends forming uniform curves. Tow size: 35 feet wide by 480 feet long, submerged 8 feet



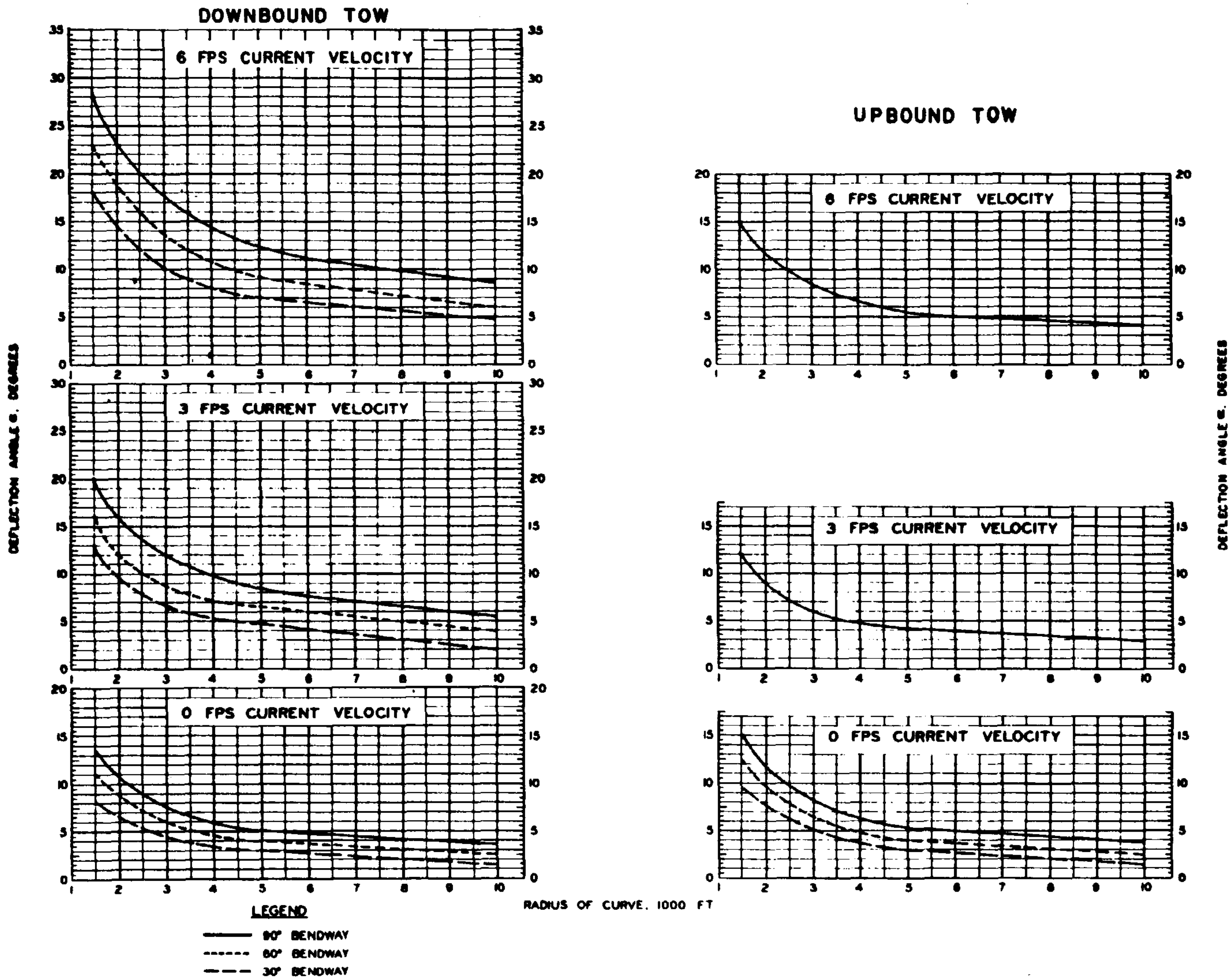
* Figure 4-7. Deflection angle for tows driving through bends forming uniform curves. Tow size: 35 feet wide by 685 feet long, submerged 8 feet



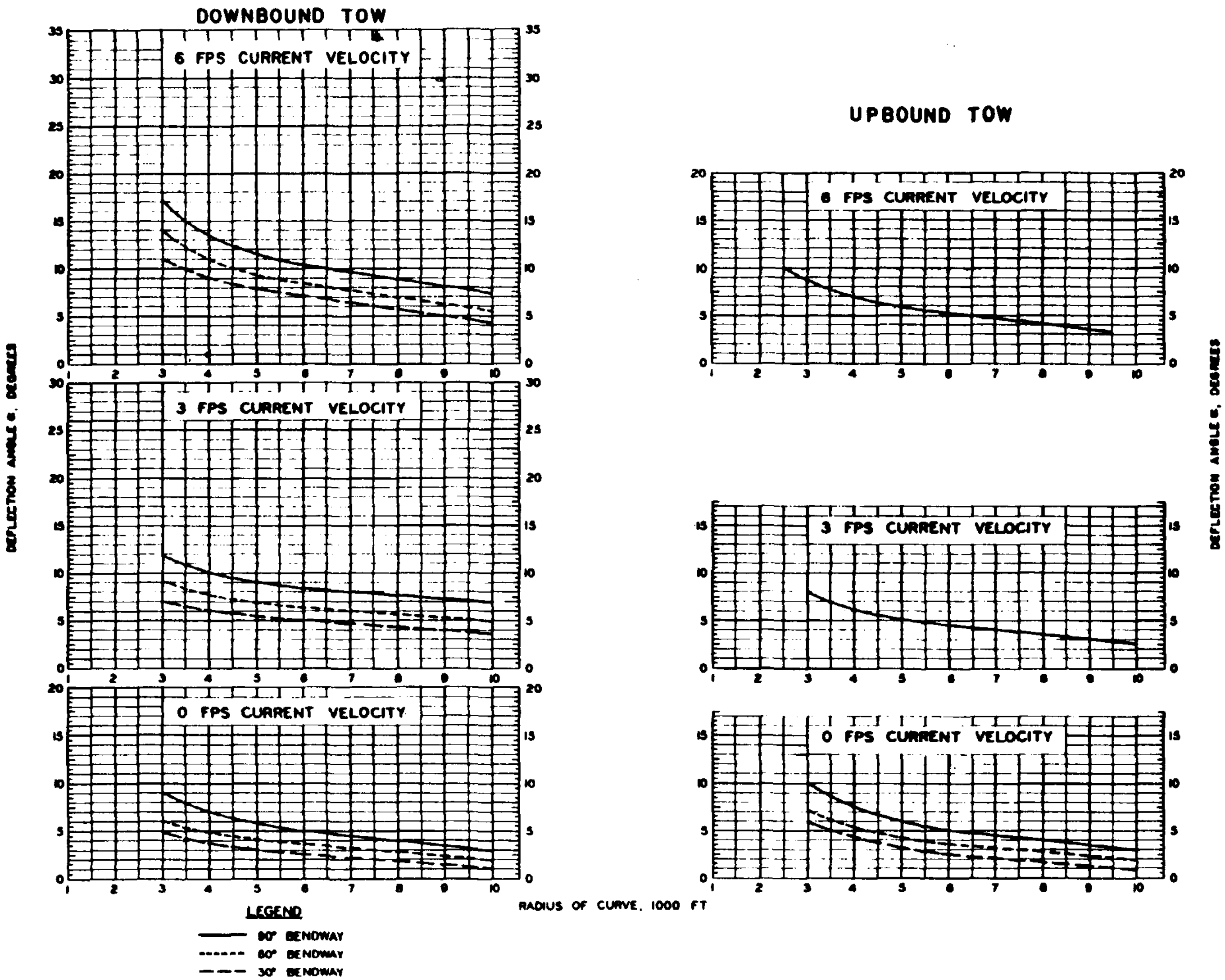
* Figure 4-8. Deflection angle for tows driving through bends forming uniform curves. Tow size: 70 feet wide by 685 feet long, submerged 8 feet



* Figure 4-9. Deflection angle for tows driving through bends forming uniform curves. Tow size: 70 feet wide by 480 feet long, submerged 8 feet



* Figure 4-10. Deflection angle for tows driving through bends forming uniform curves. Tow size: 105 feet wide by 600 feet long, submerged 8 feet



* Figure 4-11. Deflection angle for tows driving through bends forming uniform curves. Tow size: 105 feet wide by 1200 feet long, submerged 8 feet

* where bank failure or erosion of this magnitude can be expected, the bank should be protected along a selected alignment. Existing scallops might require some remedial measures such as filling in the affected areas, adding dikes to prevent or reduce the movement of currents into the areas, or increasing the width of the channels to permit tows to move a sufficient distance from the affected bank lines. *

* 4-11. Basis of Design. The design of a channel for one-way traffic has to be based on the channel width required for downbound tows. For two-way traffic, it is assumed that downbound traffic would move along the concave bank of the bend and upbound along the convex bank. The radius of the bend used for upbound traffic in the figures is that of the concave limit line, same as for downbound tows as shown in figure 4-5. The clearance between tows and between the tow and channel limit lines is a matter of judgment, skill of the pilot, and how well the limit lines are defined. During the model tests the minimum clearance between the channel limit lines and the tow was assumed to be 20 feet and between tows, 50 feet for two-way traffic. For one-way traffic a minimum clearance of 40 feet should be provided between tow and channel limit lines. Improvement of natural streams for navigation will in most cases involve some modification in channel alignment, width, and depth. In streams carrying little or no sediment, it might be more economical to increase the width of the channel in bends than to increase the radius of curvature. In streams where there is a sizable sediment load, a wider channel in bends would not be self-maintaining and could require considerable maintenance dredging.

Section V. Bridge Location and Clearances

* 4-12. Location. Numerous accidents involving collision with bridge piers have occurred on inland waterways with considerable damage to property and, in some cases, loss of life. It is important, therefore, that the location and orientation of bridges and clearances provided for navigation be such as to eliminate as far as practicable any danger of collision with the bridge structure. As a general rule, bridges should not be located in a bend, just downstream of a sharp bend, or where crosscurrents can be expected. When more than one bridge is required in a given locality, the bridges should either be close together with the piers in line or far enough apart to permit tows passing one bridge to become properly aligned for passage through the next bridge.

* 4-13. Clearances. The navigation span (horizontal clearance between piers) should be somewhat greater than the designed width of the channel in the reach depending on the alignment and velocity of currents in the reach, alignment of the channel approaching the bridge particularly from

upstream, and the probable effects of the prevailing winds. The vertical clearance should be sufficient to permit tows to clear the low members of the bridge within the navigation span at the maximum navigable flow. Bridge clearances, both horizontally and vertically, are the responsibility of the U. S. Coast Guard and planning should be coordinated with the local district of that organization.

CHAPTER 5

OPEN-RIVER NAVIGATION

5-1. General. Open-river navigation implies the use of natural streams for navigation without locks and dams, such as the Missouri River and Mississippi River below St. Louis, Mo. Except for short reaches, there are very few, if any, natural streams left in the United States that could be developed for unrestricted traffic. However, many of the factors affecting the development of open-river navigation are also applicable to canalized streams utilizing low-lift locks and dam.

5-2. Cost. The development of open-river navigation usually involves lower first cost but maintenance cost could be high because of the complex nature of these streams, tendency to meander and migrate, and difficulty of designing the training and stabilization structures needed. Operation cost of the waterway is generally small, consisting mostly of periodic surveys and inspections, channel marking, and possibly some traffic control.

5-3. Factors Affecting Navigation. Open-river navigation could be adversely affected by high-velocity currents, limited channel depth during low water, lack of suitable docking and staging areas, and constant changes in river stage and discharge. Unless their effects are considered and minimized or eliminated, navigation could be suspended for periods that could affect transportation cost to such an extent that the potential of the waterway would not be fully developed.

5-4. Feasibility Study. The feasibility study should consider all of the factors that could affect navigation and cost. This study should include analyses of the following:

a. Frequency and duration of river stages and discharges based on existing records.

b. Channel width, depth, and alignment available, particularly during low flows.

c. Composition of the bed and banks.

d. Sediment characteristics of the stream and changes produced by variations in stages and discharge.

e. Training and stabilization structures and corrective dredging required and their effects on sediment movement, currents affecting navigation, and the flood-carrying capacity of the stream.

f. Type and volume of traffic that could be developed and justified with various improvement plans.

g. Navigation periods that would be affected by unusually wet or dry years and ice.

h. Model studies required to determine problems that will be encountered and types and amount of training structures required and to develop plans for the improvement of critical reaches.

CHAPTER 6

CHARACTERISTICS OF NATURAL STREAMS

Section I. General

6-1. Natural Streams. Natural streams can be characterized by their tendency to meander and migrate, irregularity and changing geometry, varying stage and discharge, and variations in the composition of bed and banks. Because of these variations, no two reaches are exactly t same. Many of the problems encountered in the development and improv ment 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 a alignment and the operation and use of facilities and structures for navigation such as locks, harbors, docking areas, and other facilitie such as hydroplants, sewage systems, and water intakes. Sediment mov ment can also affect the capacity of the channel to pass flood flows.

6-2. Sedimentation Problems. Since sedimentation problems can affec the type of waterway that could be developed and construction and mai tenance cost, it is important that they be recognized and considered. The movement of sediment in natural streams is extremely complex depe ing on many factors, most of which are interrelated. Solution of sed mentation problems requires a knowledge of the general characteristic of the stream and of the principles of river sedimentation processes.

6-3. Sediment Load. Considerable research on the movement of sedime has led to a better understanding of the mechanics of sedimentation a to the development of theories and formulas. Most of what has been written on the subject has been based on two-dimensional flow and is general to have any practical application in the solution of most riv problems. The total sediment load of streams has been based on measurements using various sampling methods or computations using one of several available sedimentation formulas. The accuracy of measuremen would depend on the number of measurements covering a wide range of discharge and is affected by the difficulty of measuring sediment mov ing as bed load. Sediment computations are generally based on averag conditions and could be in error by a sizable amount because of the variations in the factors involved such as slope, depth, velocity, an discharge. Even if sediment measurements and computations could be made with a high degree of accuracy, a satisfactory method of using t information in the solution of most practical open-river problems has not yet been developed.

6-4. Third Dimension. The movement of sediment in a stream has to be considered in three dimensions. The third dimension is provided by the Franco principle of lateral differential in water level. This principle is stated as follows: "When conditions are such that a lateral differential in water level (or transverse slope) exists or is produced by changes, there will be a tendency for at least some of the total flow to move toward the lower elevation; the slower moving, sediment-laden bottom currents can make the change in direction easier than the faster moving surface currents and account for the greater concentration of sediment moving toward the lower elevation." This general principle is involved in many of the developments in alluvial streams including the development of sandbars on convex side of bends, movement of sediment around the end and behind dikes, development of cutoffs and divided channels, shoaling in lock approaches, etc. In each case there is either a buildup in water level on one side or a reduction caused by channel enlargement, contraction, or flow diversion that causes some of the flow to change direction.

Section II. Shoaling Problems

6-5. Deposition. 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 in long flat bends 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 a 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.

6-6. Stage and Discharge. Changes in the discharge and stages produce changes in currents and in the movement of sediment that render the application or development of design principles extremely difficult. Model and field investigations have indicated how channel depths and configurations can be altered with change in stages. The movement of sediment in one reach can be considerably higher than in a reach just downstream during low flows and considerably lower during high flows.

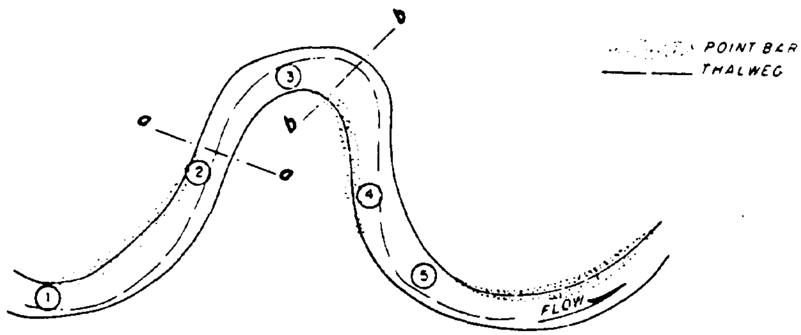
When one reach is not capable of moving the entire sediment load, shoaling will occur in that reach until velocities, slopes, and carrying capacity of the channel increase to that required to move the load.

6-7. Low-Water Profiles. Changes in low-water slope profiles are usually indications of the relative amount of sediment movement in successive reaches. When the low-water slope in a reach is substantially higher than the average, it is generally an indication that more sediment was moved into that reach from upstream during the higher flows than could be moved through the reach during the same and subsequent flows. Unstable and troublesome reaches will tend to have a higher-than-average low-water slope.

6-8. Meandering Channels. Natural streams 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 streams depends on many factors including discharge, sediment load, valley slope, and composition of bed and banks. Unless the meandering of these streams is resisted by 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 channel is deeper in bends along concave banks and shallower in crossings and straight reaches (fig. 6-1).

6-9. Scour in Bends. The channel in bends tends to deepen during high river stages. Scour generally starts near the upper end of the bend and progresses toward the downstream as discharge and river stages increase. 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. Where depths increase with river stage, shoaling of the channel starts during the falling stages near the upper end of the bend and continues toward the downstream during the low-water period.

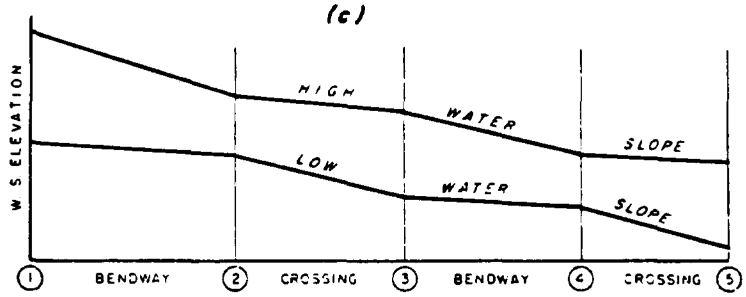
6-10. Sediment Movement. The scouring of the channel in bends can cause a large amount of sediment to move into the crossing and reach just downstream. Because of the concentration of high-velocity currents and turbulence in bends, much more sediment can be moved in sinuous channels than can be moved 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



DIAGRAMMATIC PLAN OF RIVER BEND
(a)

SECTION THRU (a - a)
(b)

SECTION THRU (b - b)
(c)



WATER-SURFACE SLOPES IN BENDWAYS AND CROSSINGS
(d)

Figure 6-1. Characteristics of a river reach backwater effect produced by the shallow crossing downstream (fig. 6-1d). Because of the reduced slopes and velocity, deposition occurs in bends during low flows. However, the amount of deposition is seldom sufficient to reduce depths to less than that required for navigation.

6-11. Crossings. In meandering streams the low-water channel in the straight reach between alternate bends crosses from one side of the river to the opposite side (fig. 6-2). 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 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 and stage duration. After a prolonged high-water period or after a rapid decrease in stage, depths over crossings will tend to limit channel 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.

6-12. Straight Channels. Channels in long straight reaches or in long flat bends will tend to meander within their banks and be unstable and troublesome. 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 and troublesome reaches will tend to have a higher low-water slope than will stable reaches.

6-13. Divided Channels. Bifurcated channels or divided flow will be found in many alluvial streams in addition to those formed by cutoffs. Side channels will tend to carry a greater proportion of the sediment load than the proportional discharge, because of the lateral differential in water level which depends upon 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

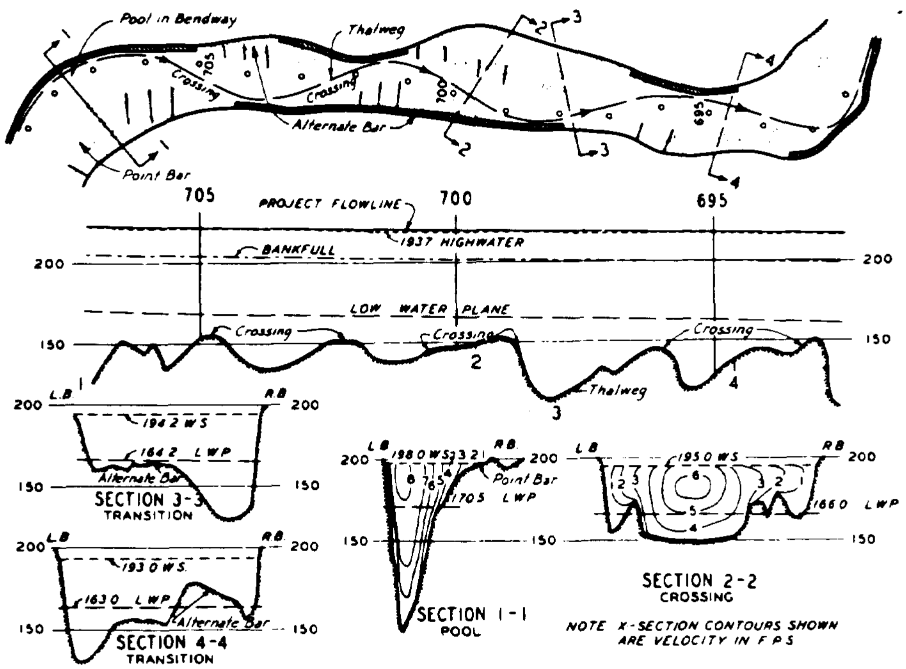


Figure 6-2. Features of a typical improved
reach--Mississippi River

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 (fig. 6-3).

6-14. Tributary Streams. 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 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 (fig. 6-4).

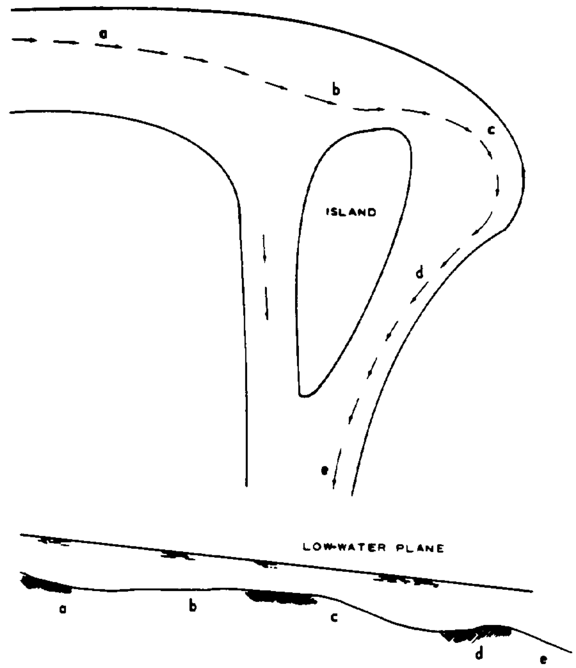


Figure 6-3. Typical divided channel

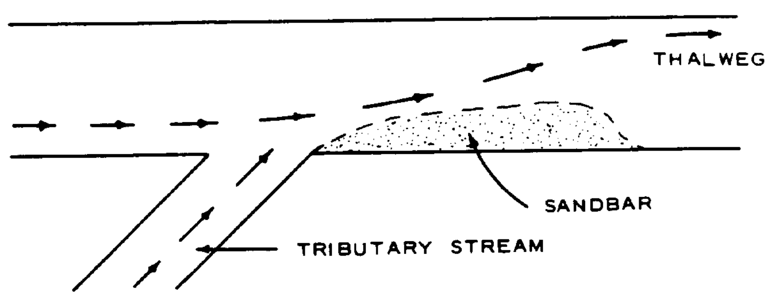


Figure 6-4. Effect of tributary on channel configuration

6-15. <u>Entrances to Canals and Harbors.</u> Entrances to canals and slack-water 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 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 which cause flow toward and away from the canal or harbor (fig. 6-5).

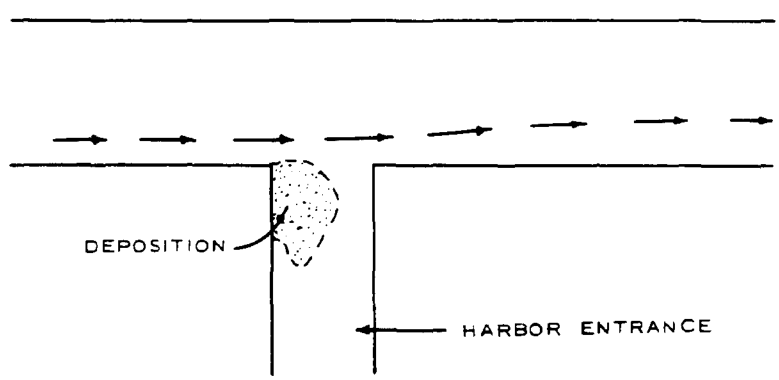


Figure 6-5. Shoaling in harbor entrance located along riverbank

CHAPTER 7

IMPROVEMENT OF NATURAL STREAM CHANNELS

Section I. General

7-1. Requirement. The improvement of natural streams for navigation involves channel realignment, stabilization, training structures, and in many cases the modification or replacement of existing bridges. In streams carrying large quantities of sediment, a sinuous channel is easier to develop and maintain than channels in long straight reaches or long flat bends and should be considered in the layout and planning for the project. The sinuosity of a stream varies over a wide range. However, design should be based insofar as practical on the alignment of reaches that have been reasonably stable with a channel adequate for the traffic anticipated. Channel realignment will be required to eliminate or reduce the curvature of sharp bends and the tendency for shoaling. Channel realignment involves corrective dredging, training and stabilization structures, or in some cases cutoffs.

Section II. Dredging

7-2. Corrective Dredging. Corrective dredging is used to realign the channel or bank lines and to develop cutoffs. Dredging in the channel bed involves the removal of erosion-resistant material such as gravel bars, rock outcrops, or clay plugs. Usually dredging within the channel bed without some training or contracting structures will produce only temporary results and might have to be repeated after each high-water period or significant rise in river stages.

Section III. Channel Stabilization

7-3. Bank Erosion. Channel stabilization involves the protection of the banks of streams or canals from erosion caused by currents or wash from waves created by wind and traffic. Natural streams with erodible bed and banks will tend to meander and migrate and, unless this tendency is resisted, will be constantly changing. Erosion of the channel bed along a bank will tend to undermine the bank or steepen its slope to the point that caving or sloughing of the bank occurs. Erosion and caving of banks can adversely affect channel alignment and depth, can increase sediment load and maintenance cost, and could result in the loss of valuable land and endanger local installations such as buildings, rail lines and highways, bridges, docking facilities, and flood-control levees or floodwalls.

7-4. Types of Protection. Bank protection can be a major cost in the development of a waterway for navigation and should be considered during the initial planning of the project. Some of the cost might be considered as part of the flood-control aspects, particularly if it is a multipurpose project. The type or types of bank protection vary depending on the characteristics of the stream, particularly the variations in stage and discharge and the erodibility of the streambed and streambanks. Bank protection and stabilization might consist of structures such as dikes designed to divert currents away from the bank or improve the alignment and velocity of currents along the bank. The most common type of bank protection is some type of revetment covering the bank and channel along the toe of the slope with erosion-resistant material or blanket. In canals with no currents and water level maintained reasonably constant, only a small section of the bank above and below the water line is normally required for protection against wave action. The type of revetment used should be based on experience on waterways of the same general characteristics and construction and maintenance cost.

Section IV. Cutoffs

7-5. Purpose and Method. Cutoffs are used to eliminate sharp bends, eliminate troublesome reaches, reduce the length of the navigation channel, or increase the flood-carrying capacity of the stream. Cutoffs are usually formed by dredging a pilot channel across the neck of one or more bends. The size, slope, and alignment of the pilot cut should be such that the cutoff 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 is made, 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. The rate of development of a cutoff can be increased by the gradual closure of the old bendway channel or by structures designed to increase the tendency for shoaling in the upper end of the existing bend and to direct flow toward the pilot cut.

7-6. Old Bendways. In planning cutoffs, the use of the old bendway for recreation and/or harbor facilities should be considered. In many cases, the general practice has been to close off the upper end of the old bend with a closure dike or embankment to eliminate the movement and deposition of sediment in the bend. Structures will usually be required in the lower end of the old bend to reduce the tendency for shoaling and the need for maintenance dredging (fig. 7-1).

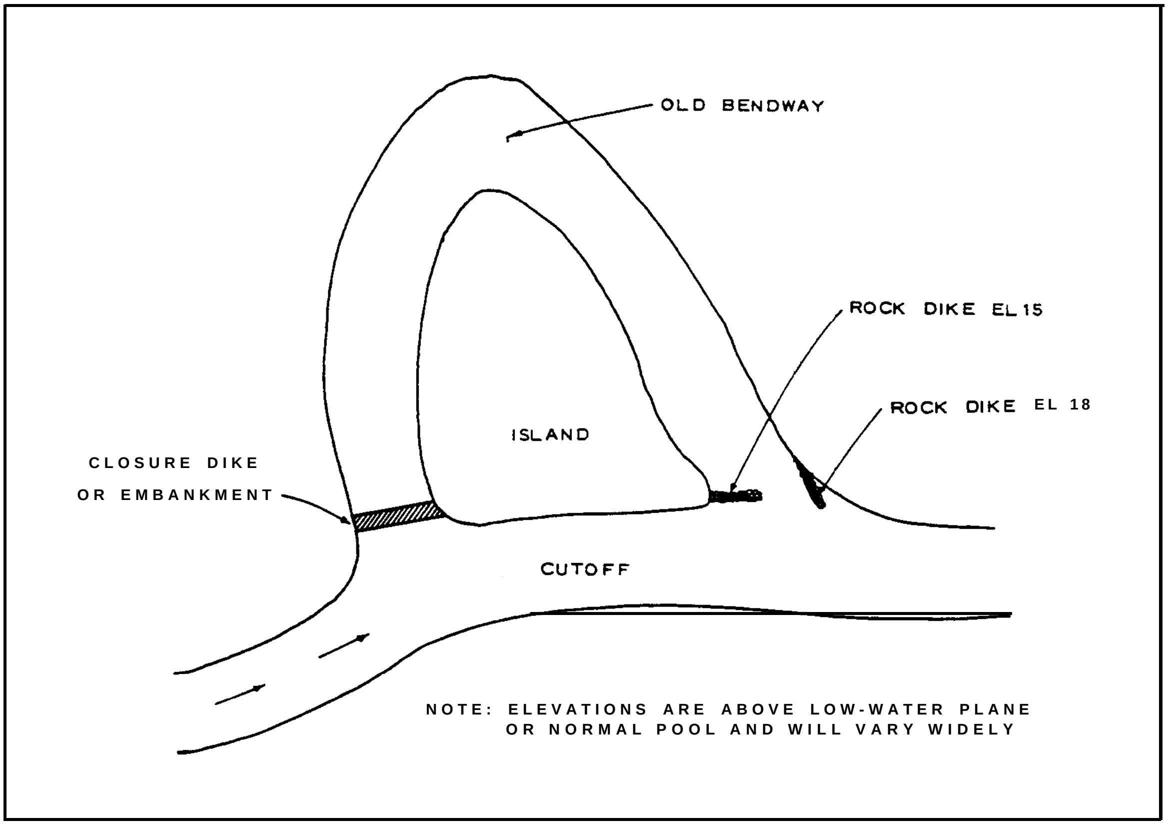


Figure 7-l. Cutoff-old bendway used for harbor or recreation

* ***Section V***
Training Structures

7-7. Introduction

The development and improvement of the alignment, width, and depth of a navigation channel often require the use of river training structures to contract the channel, thereby temporarily increasing velocities and corresponding river depth. Training structures can also be used to realign the channel and stabilize the location of the low-water channel. These structures usually consist of some type of dikes constructed of stone. Historically, dikes were constructed of timber pile clusters or piling with a stone fill. These structures tended to have high labor costs due to the intensive manpower required for construction. Pile dikes tended to fail due to accumulation of drift or ice, rotting, corrosion of bolts and ties, lack of maintenance, and fire. For these reasons the use of pile dikes is very limited. The type or types of structures used and their configurations should be based on the characteristics of the stream, problem or problems to be resolved, and conditions contributing to the problem The design of the structures should consider the effects of the structures on currents existing in the reach, the movement of sediment, and the effects of the resulting currents on navigation.

7-8. General

The use of dikes on navigable rivers meets the need for permanent structures with a low maintenance cost The dikes are effective at confining the river in a single channel, with the goal of providing depths suitable for commercial navigation at the full range of expected flows. Also, the use of dikes minimizes or eliminates dredging for channel maintenance. Channel dredging is only a temporary measure, and dikes can significantly reduce the need for maintenance dredging. Dikes function continually at all river stages and concentrate the river's energy into a single channel to control the location and depth of the navigation channel and impact the erosional and depositional characteristics of the river. The dikes not only must be capable of controlling the low-water navigation channel, but should not unduly restrict the flood-carrying capability of the river. 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, spurs, * and jetties. All of these names typically apply to a river training structure that is approximately normal to the riverbank, is attached to the river bank, and contracts the natural river channel but does not transverse the entire river channel. Other types of river training structures include longitudinal dikes, L-head dikes, vane dikes, and bendway weirs or submerged sills (Figure 7-2).

7-9. Spur Dike Design Parameters

A spur dike is defined as a structure placed approximately perpendicular to the bank line to concentrate the flow into a single channel. **The** design of spur **dies 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, and method of construction (Figure 7-3).

7-10. 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- The amount of contraction should not unduly increase flood heights, and the channel should be capable of carrying the sediment load associated throughout the full range of flows. The limits of contraction, called channel controllines or rectified channel lines, are normally determined through a combination of experience, use of model studies, and qualitative 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 and integrated into the composite design. The qualitative analysis method is normally 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.

7-11. 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 7-4). The contracted channel width is established on

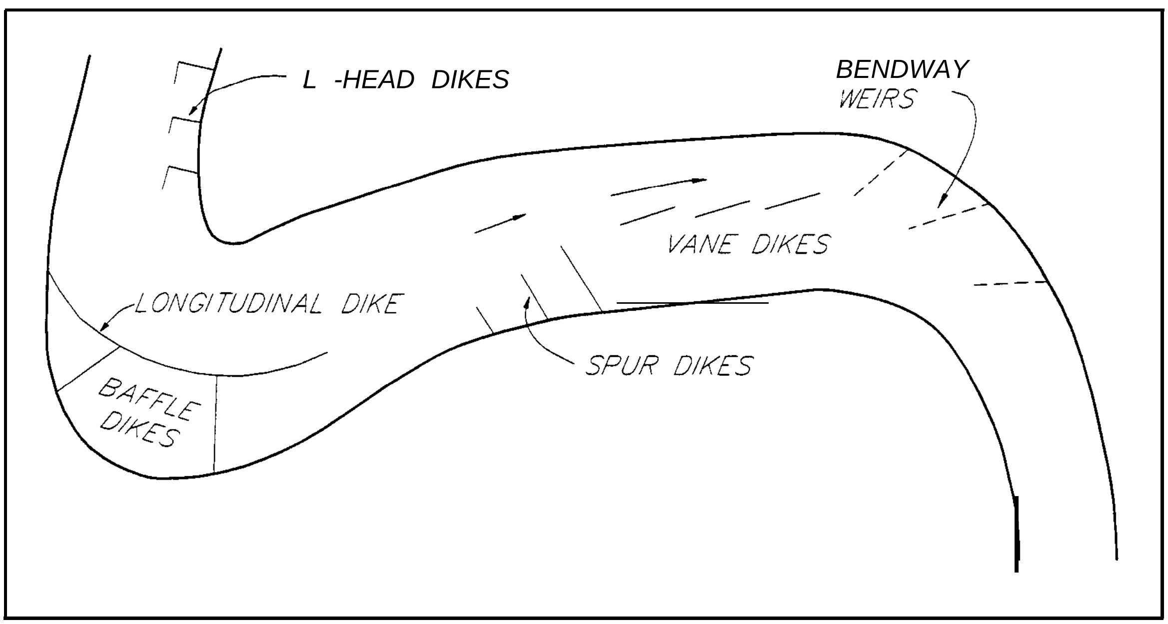


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Figure 7-2. Types of training structures in use

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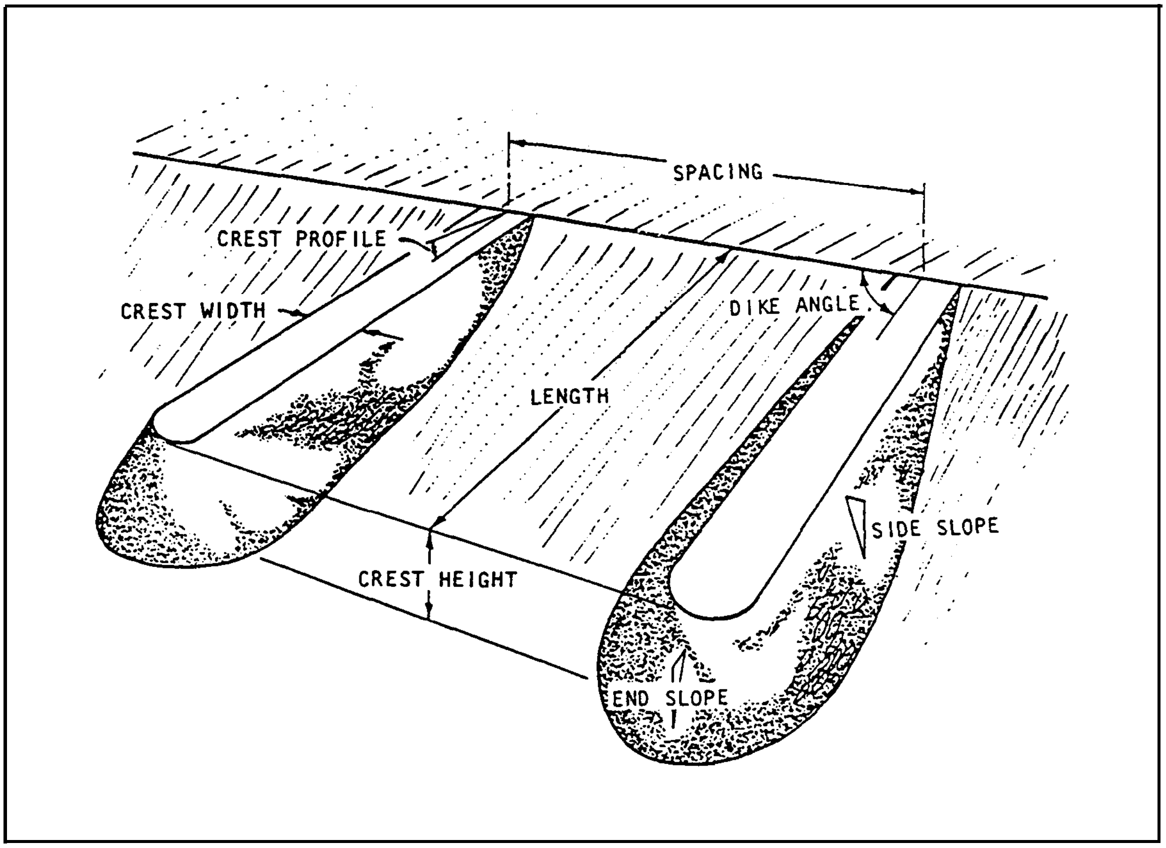

Figure 7-3. Stone spur dike parameters

the map with the channel control lines providing the designer with the right and left channel limits. This map is in essence 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 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, dredging maintenance costs will be minimized or entirely eliminated. The design should ensure that crossing lengths are not too long, which may encourage the development of middle or alternate bars within the channel (Figure 7-5). 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. The designer should make every effort to ensure that the final layout is compatible with the existing natural channel layout and that realignments fit within these limits. 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.

7-12. Dike Length

The length of spur 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 river-ward of the channel control line was at a significantly blower elevation than the main portion of the

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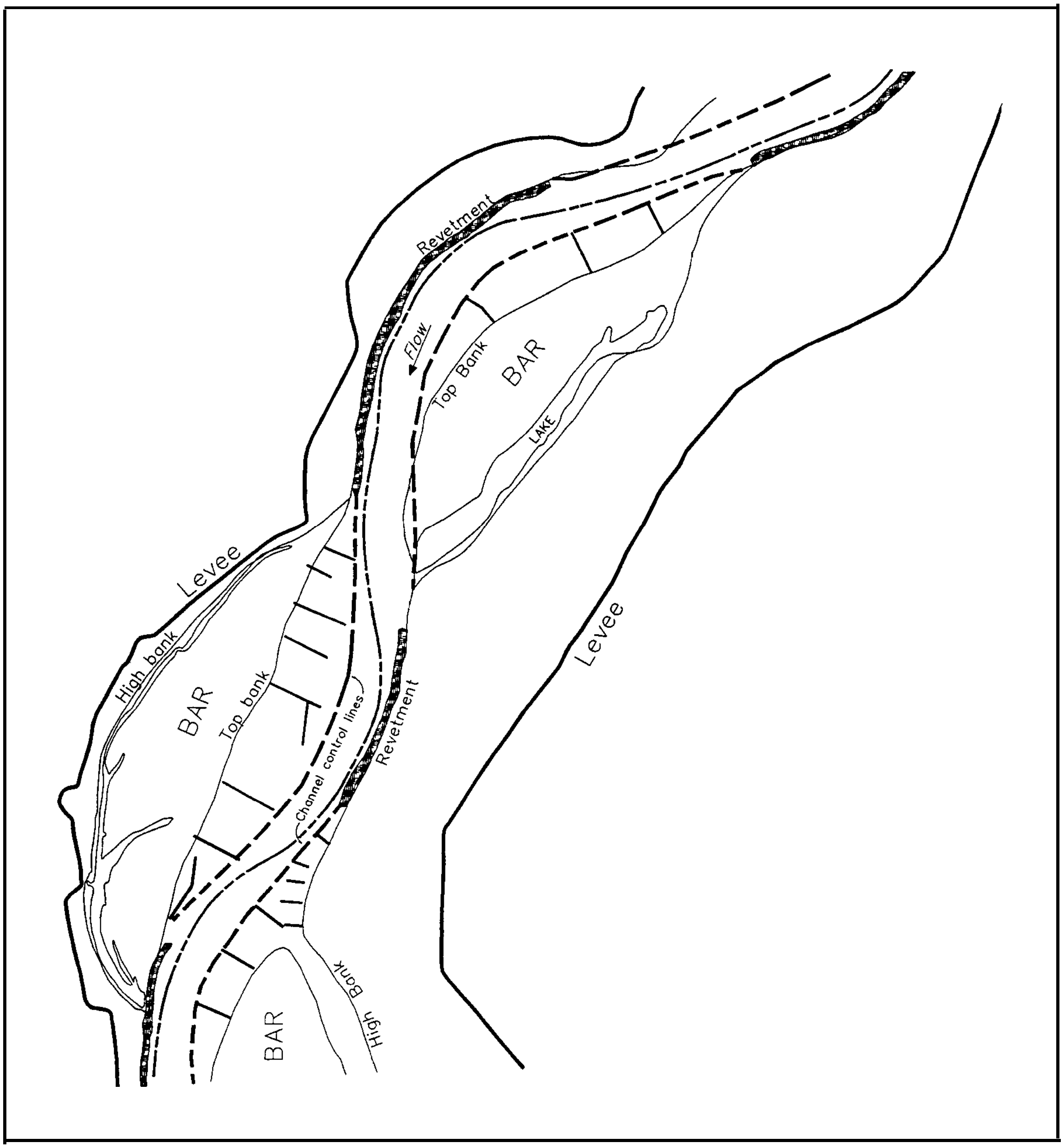

Figure 7-4. Example of layout for channel control lines

dike. Providing dike lengths to the desired channel control lines is adequate for initial construction; however, once the stream has reacted to the dikes, modifications and adjustments may be required.

7-13. Dike Height

The height or top elevation of dikes is normally associated with a sloping reference plane parallel to the water surface through the project. In open river projects the reference plane may be called Annual Low-Water Plane (ALWP), Low-Water Reference Plane (LWRP), or Construction Reference Plane (CRP). In canalized waterways the reference plane is normally called the upper pool elevation upstream of the dam and lower pool elevation downstream of the dam. The elevation of the dikes relative to the water surface can have an important bearing on the structure

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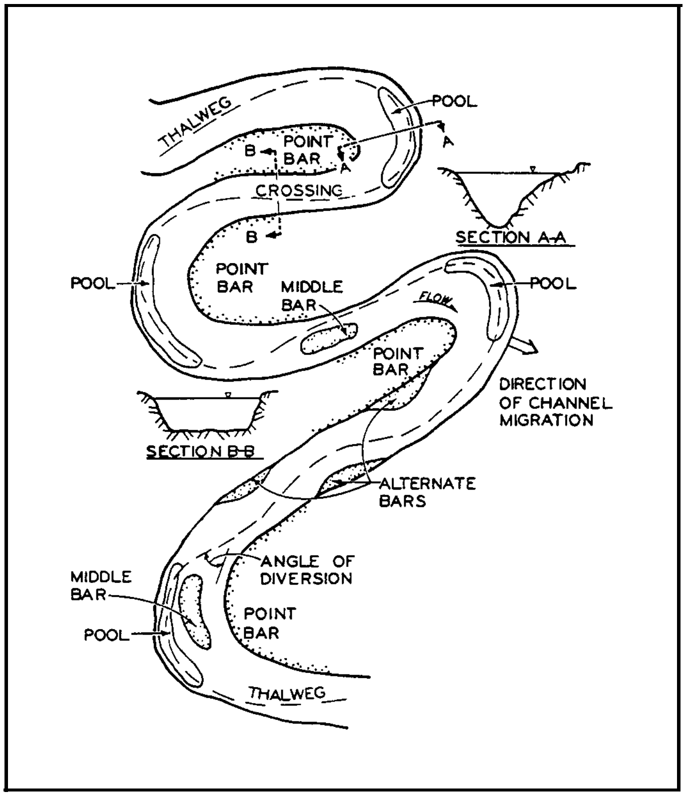

Figure 7-5. Channel planform

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 varies from about 10 to 15 feet above the reference plane. Normally this puts the height of the dikes approximately at the elevation of the midbank. On the Missouri River, au 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 then 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 normal 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 au open water area between the deposit and the original bank line. On canalized projects the top elevation of dikes is referenced to the normal pool elevation, the minimum regulated pool elevation or some similar reference. In 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. That elevation is a minimum to ensure that pilots can see the dikes and be aware of the existence of river training structures in that location.

7-14. Adjacent Dike Heights

The relationship between the height of adjacent dikes in a system, three dikes or more, is also of importance. 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
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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.

7-15. Crest Profile

The crest profile of spur 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. 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 7-6) 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.

7-16. Crest Width

The width of the crest of a spur 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 10 feet, while those constructed by truck have a crest width of 10 to 14 feet. Experience has shown in river reaches susceptible to ice flows that dikes with crest widths of less than 6 feet will 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

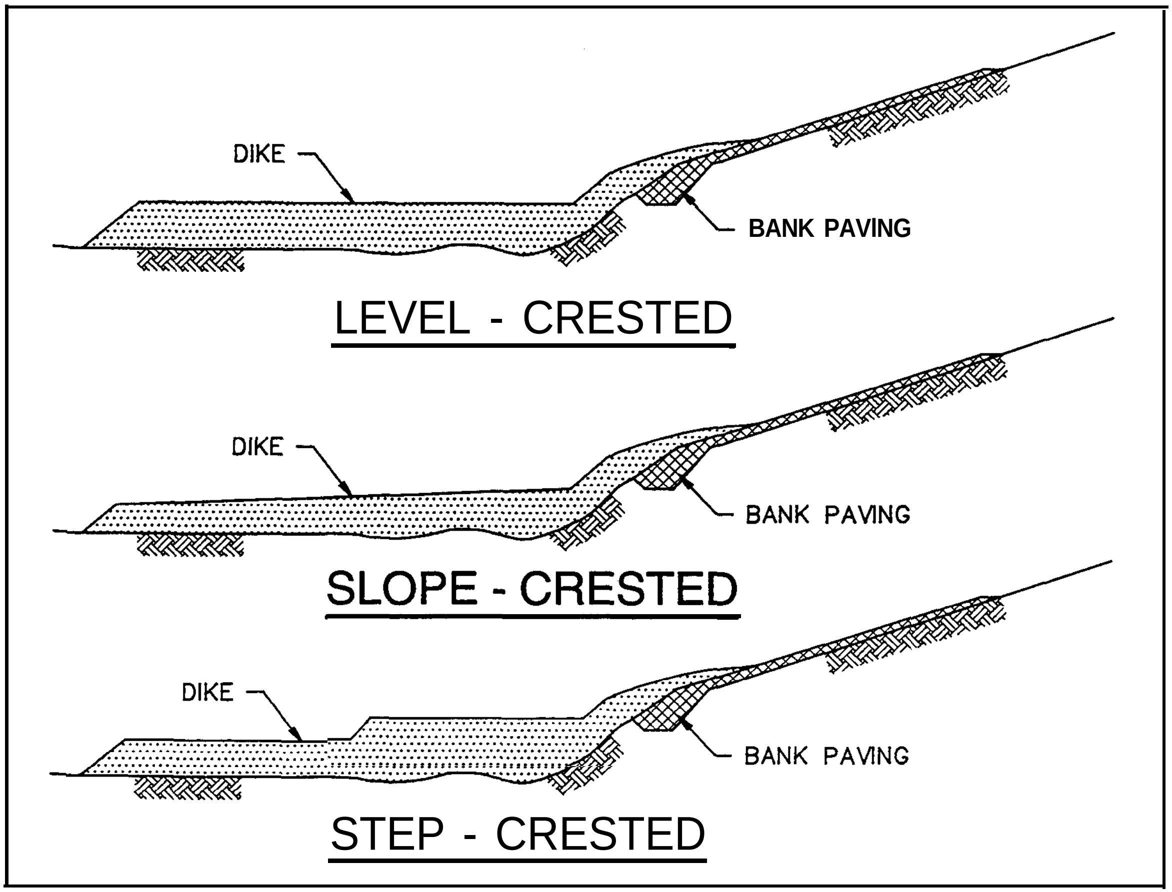

Figure 7-4. Stone spur dike crest profiles

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will produce a gap in the dike. This gap could cause scour and possible breaching of the dike with the dike becoming 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 spur dikes, but virtually all dikes fit withintherangeof 5 to 20 feet with the majority of dikes constructed with a crest width of 5 to 10 feet

7-17. Side Slopes

The side slopes (upstream and downstream faces) of spur 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 spur dikes, including computation of required stone quantities, is 1V on 1.25H to 1V on 1.5H with the difference being a function of the particular stone, the dike height, and the velocities and depth of water that stone has to fall through during construction.

7-16. End Slopes

Although the slope of the stream end of a 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. 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 die. Often an end slope of 1V on 5H is used where significant scour is anticipated, but in some unusual and severe scour circumstaucesanendslopeofasflatas 1V on 10H has been used.

7-19. 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. Model tests conducted over the years indicate that when dikes are angled upstream the scour at the end of the dike will be greater and the adjacent channel will be farther from the dikes than systems that are normal or angled downstream. Dikes angled downstream are as effective as those normal to the bank; however, care must be taken with dikes angled downstream to take into account possible flanking of the bank end of the dike. In most applications dikes are constructed normal to the adjacent bank line or angled slightly downstream, about 10 or 15 degrees. The approach with such systems is that the angled dike generally reduces the attack on the entire system

7-20. Dike Spacing

The spacing of dikes within a system should be great enough that the least number of dikes (and least stone) 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 cau also be increased. Experience has shown that a spacing of two-thirds of the length of the upstream dike produces a system that is effective. On streams with dike lengths of about 1000 feet, spacings of 1-1/2 to 2-1/2 times the length of the upstream dike have been used. However, on larger streams, 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 spur 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 convex side (outside) 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.

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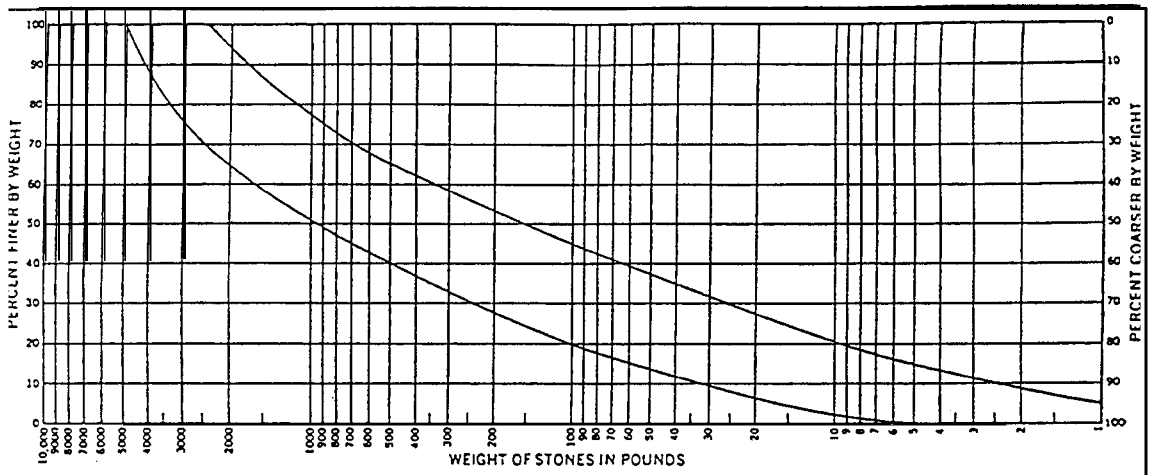

STONE WEIGHT (LBS)	CUMULATIVE % FINER BY WEIGHT
5000	100
2500	70-100
500	40-65
100	20-45
5	0-1 5
1	o-5

* ASSUMING STONE SHAPE MIDWAY BETWEEN A SPHERE AND CUBE

U.S. ARMY ENGINEER DISTRICT, VICKSBURG
CO.., OF ENGINEERS
VICKSBURG MISSISSIPPI

GRADATION
GRADED STONE A

Figure 7-7. Quarry-run stone gradation curve

7-21. Stone Size

The stone used on spur dikes is normally classified as quarry-run stone, which has a size variation depending on the particular stone available in the area. The larger stones are used to cover the surface of the spur dike and the smaller stones are used on the internal section. On the Lower Mississippi River, quarry-run stone has 5 percent by weight passing a 1/2-inch screen, 10 percent less than 5-pound pieces, 50 percent between 400 and 1,000 pounds, and no pieces larger than 5,000 pounds- Figure 7-7 is the gradation curve used for contracting for quarry-run stone on the Lower Mississippi River. On smaller navigation projects such as the Arkansas River, the stone gradation used for spur dikes has 5 percent fines, 50 percent larger than 40 pounds, and a maximum size of 1,000 pounds. On the Missouri River, the stone used in the original construction of dikes was classified as pit-run stone with a 2,000-pound maximum size on the lower two-thirds of the structure and 500 pounds maximum for the top one-third of the structure and paving of high banks. The stone presently used on the maintenance of the Missouri River dikes has 100 percent lighter than the range of 393 to 954 pounds, 50 percent lighter than 197 to 252 pounds, and 15 percent lighter than 62 to 146 pounds. This gradation is partially a function of the improvement in construction techniques and the availability of a commercial quarry for the stone. The key for selection of the particular stone to use in the construction of the dike is the availability of material and obtaining the most reasonable price to minimize the cost. A smaller percentage of fines in the stone is acceptable if obtaining such stone is the most economical. Some of those fines may be lost during actual construction, but such losses are expected and can be taken into account when ordering the stone.

7-22. Bank Paving

The paving of the bank adjacent to the bank end of the spur dike may be required to control scalloping of the bank line and possible flanking of the structure. The riverbanks on the Mississippi River are typically paved or revetted 100 feet upstream and 200 feet downstream of a spur dike. On some smaller rivers, such as the Arkansas and Missouri Rivers, bank paving upstream and downstream of the dike

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typically varies from 50 to 75 feet. Some bank grading may be required, but essentially the same procedures and methods incorporated on the stream for bank protection using revetment should be incorporated in the bank paving for a dike. If the bank end of a dike is terminated in an easily erodible material, a trench fill should be used to key the dike into the bank. This will control possible flanking of the spur dike by ensuring that the bank material remains in place during high overbank flow events.

7-23. Method of Construction

The preferred method of construction is to construct the dike in lifts for the entire length of the dike. The lifts are normally about 4 or 5 feet in height. The advantage of tbis method is that it limits any abrupt contraction of the channel and possible scour of the bed as the dike is being constructed. A die constructed to its design crest height and length in a single stage will gradually complete the channel contraction from the bank end, but may cause excessive scour at the end of the spur dike as construction is under way and result in excessive material costs. Building spur dikes in stages (or lifts) over several construction seasons will take advantage of the sedimentation that occurs downstream of the spur dike. This is a very cost effective method of construction if, during those construction seasons, a reduced effectiveness of the dike is acceptable. Dike construction can be accomplished using land-based equipment or river-based floating plant Where permissible, construction from a floating plant is often more cost-effective, as transportation of construction materials by barge is usually the least expensive. Model studies have indicated that a preplanned construction sequence may be beneficial when laying out a dike system, where the most upstream dike or dikes are constructed first, then the river is allowed to react to this construction prior to installing the next downstream structures. This procedure assists the designer in determining the most desirable structure spacing and orientation, and also can have cost benefits by taking advantage of newly formed deposits as a result of the upstream construction, thereby reducing the required volumes of dike material. Disadvantages include an extended construction period and delays in achieving the required channel dimensions.

7-24. Longitudinal Dikes

Longitudinal dikes are continuous structures extending from the bank downstream generally parallel to the alignment of the channel being developed (Figure 7-2). Properly designed longitudinal dikes are the most effective type of structure in developing a stable channel since such 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 7-8 shows the application of a longitudinal dike on the Arkansas River. It should be noted that the tie-in dikes

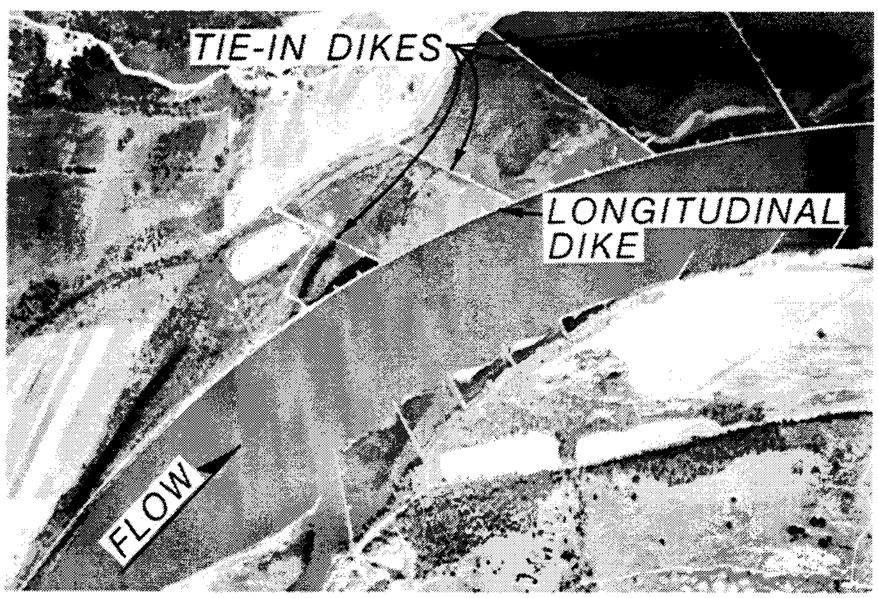

Figure 7-6. Longitudinal dike on Arkansas River

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* landward of the longitudinal dike add stability to the entire structure. These tie-in 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.

7-25. Vane Dikes

Dies 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 7-2). These dikes consist of segments of dikes located riverward from the existing bank with gaps between the dikes (Figure 7-9). 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 spur 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. Figure 7-10 is au example of a vane dike system on the Mississippi River. On some of the vane dike systems that have been in place for many years some of th vanes have been connected to the bank line with a spur dike creating an L-head dike. This modification was undertaken after significant shoaling of material between the vanes and the area landward of the dikes had taken place.

7-26. L-Head Dikes

L-head dikes are spur dikes with a section extending downstream from the channel ends generally parallel to the channel line (Figure 7-2). The addition of the L-head section can be used to reduce the spacing between spur dikes, to reduce scour on the stream end of the spur dike, or to extend the effects of the spur dike system farther downstream. L-heads tend to block the movement of sediment behind the spur dike. When the L-head crest is lower in elevation than the spur 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 7-l 1 is an example of use of L-head dikes on the Mississippi River to reduce the effects of a major bank line discontinuity. In such an application a longitudinal dike would have been effective also; however, it is obvious from the photograph that use of L-head dikes was probably as effective, but at a much reduced cost due to significantly lower quantity o.

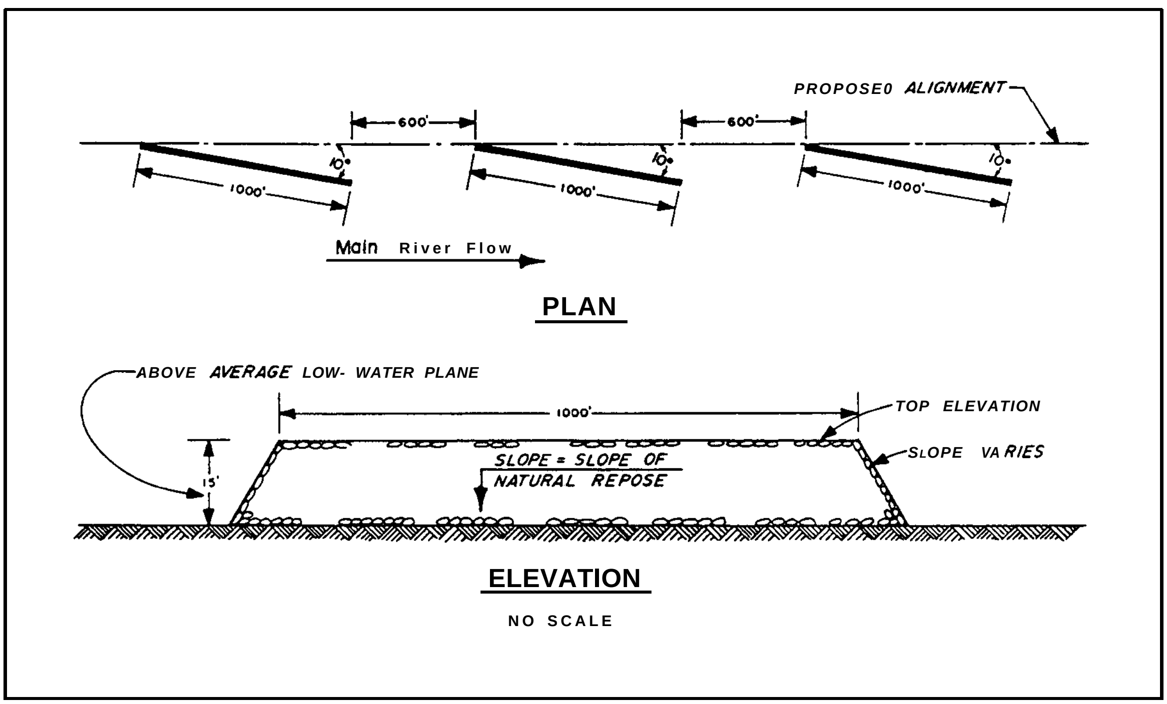

Figure 7-9. Vane dike layout

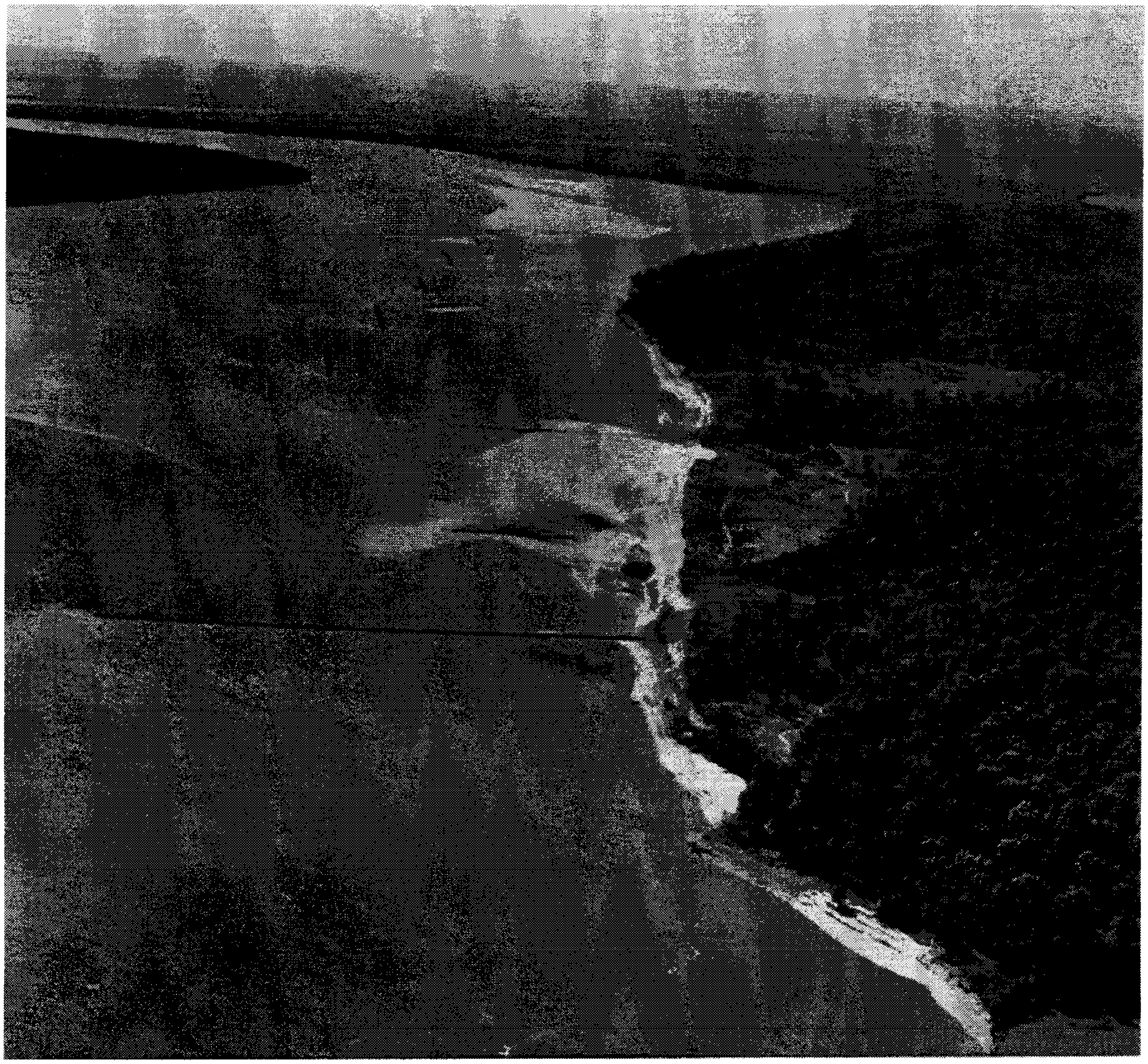

Figure 7-10. Vane dike system on the Mississippi River near Greenville, Mississippi

stone required to construct the L-head dikes verses a longitudinal dike.

7-27. 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 7-12).

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Figure 7-l 1. L-head dikes on the Mississippi River

7-28. Dike Notches

In recent years notches or environmental gaps have been added to new or existing dike systems to preserve open water areas for channel conveyance and environmental enhancement. The notches have created habitats that seem to be relatively large surface areas of quiet, slack water during medium to low river stages. This has been accomplished with apparently little adverse impact on the primary purpose of the dikes to maintain adequate navigable channel depths on the project The notches add to the wetted perimeter within the dike field, which helps diversify the habitat. These notches are created by removing stone from existing dikes, leaving notches during repair of damaged dikes, or designing notches in a new dike. The notches typically are constructed with a triangular or trapezoidal section with lengths varying from 20 to 100 feet (along the axis of the spur dike) and depths of 3 to 12 feet below the crest elevation of the dike (Figure 7- 13). Notches installed in closure dies have also been helpful in maintaining a limited amount of flow through secondary channels as habitat enhancement in those areas.

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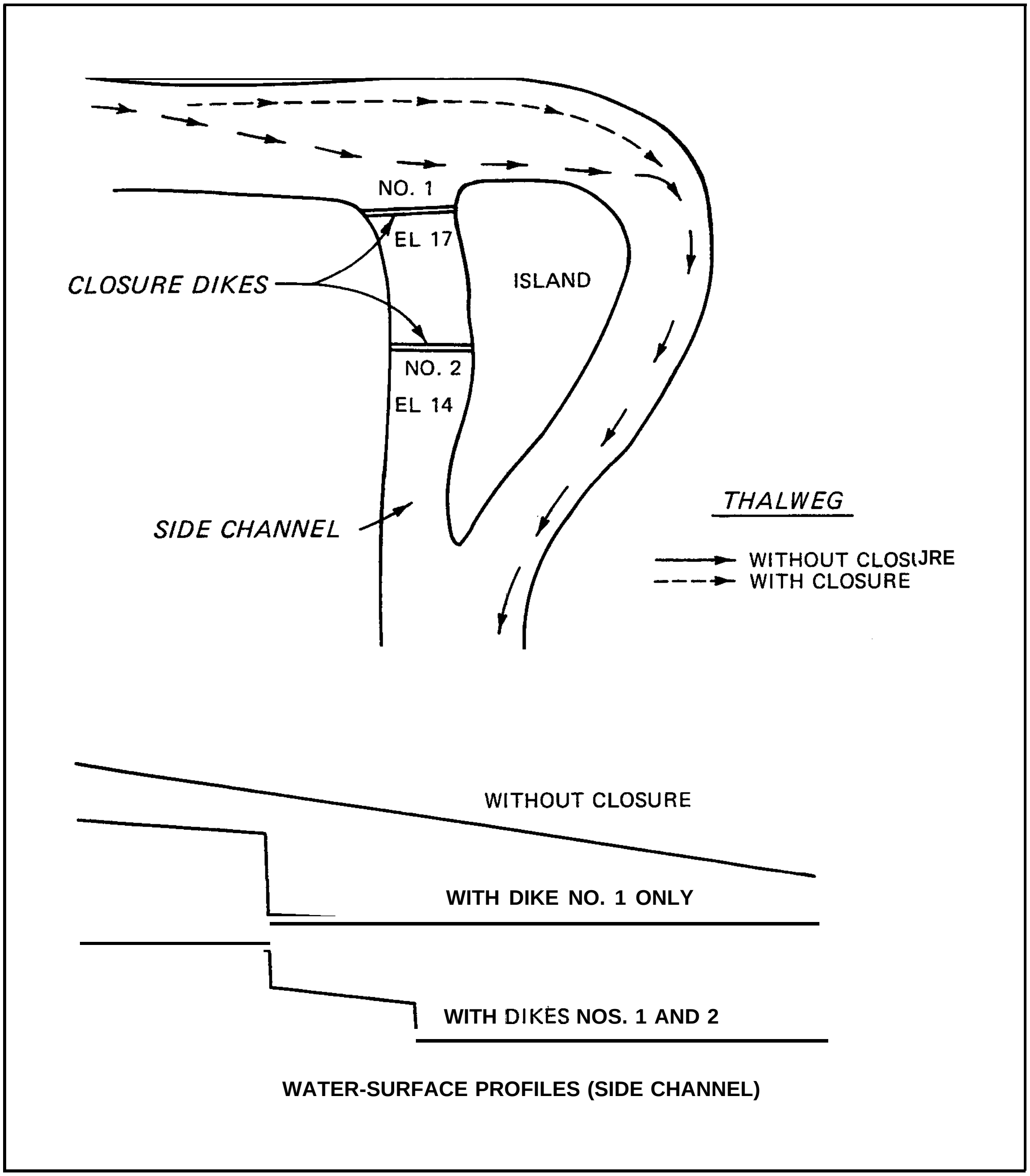

Figure 7-12. Side channel closure

7-29. Bendway Weirs

A recent development in river training structures involves the concept of bendway weirs to modify the channel in bendways. Typically the natural riverine processes will create a point bar on the inside or convex side of the bend (Figure 7-5). 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 at au elevation of 15 to 20 feet below the record low water for that portion of the *

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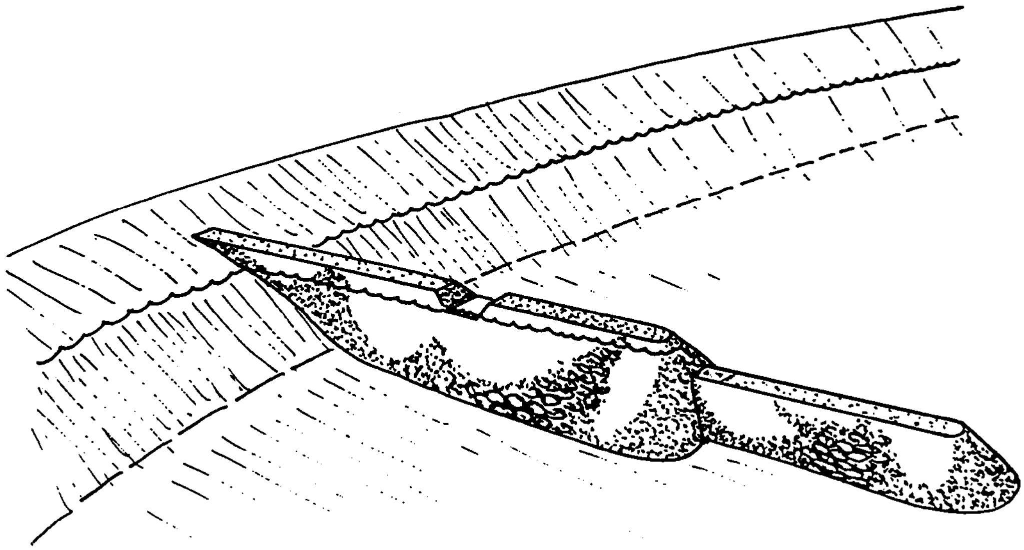

Figure 7-13. Spur dike with notch

river. The concept was model tested and developed on a portion of the Mississippi River upstream of the confluence with the Ohio River. Since being constructed in the prototype, the bendway weirs have performed exceptionally well with significant reduction in maintenance dredging quantities in a short period of tune, significant improvement in navigation channel width, increased bank line stability of the concave bank, and improvement in the crossing downstream of the weirs. The concept of installing submerged weirs or sills is not new to river training (paragraph 10-4). Previously model tests were conducted to investigate underwater sills on the convex bank of a Missouri River bend, but this new application on the concave bank in bendways and angling the structures upstream is an innovative approach in river training structures. At this time no general design guidance is available for inclusion in this manual; however, research is presently being conducted to develop appropriate design parameters. As the appropriate data are analyzed and reviewed, the design parameters for bendway weirs will be included in future manual updates.

7-30. Benefits of Bendway Weirs

Model study results indicated that installation of bendway weirs in a problem reach could potentially provide additional benefits beyond the desired one of increasing the width of the navigation channel in the bend. Since the weirs are located on the outside (concave bank) of the bend in the deep portion of the channel, some of the secondary currents that tend to concentrate flow along the outside of the bend arc broken up. This improves the flow conditions through the bends: high-velocity currents are no longer concentrated on the outside of the bend, thus the resulting currents are more evenly distributed across the channel. These lower, more evenly distributed currents make navigation conditions safer and more efficient for the towing industry. The redistribution of the currents also allows bed material to accumulate on the outside of the bend in the deep portion of the channel, which adds stability to the bank line. Tests also indicated that there may be an improvement in the navigation channel immediately downstream of the reach with bendway weirs. This change is a result of the redistribution of water and sediment in the bendway and how it now approaches the downstream reach. The fourth additional benefit may be realized by an environmental improvement in the habitat. The weirs may act like reefs, drawing lower members of the food chain and ultimately fish. The widening of the navigation channel provides a wider, shallower channel and a more usable fishery habitat. Elimination of the encroachment of the

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point bar into the navigation channel will reduce the need for maintenance dredging and the associated dredged material disposal problems. The majority of these benefits observed or concluded from the model studies have also been demonstrated in the prototype application of the bendway weir concept.

7-31. Maintenance Requirements

All dikes, regardless of the specific design or construction material used, will require periodic maintenance in order to ensure their structural integrity and intended purpose. 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. One must never lose sight of the purpose for which the dikes were built, and if the dikes are successful in maintaining the desired navigation channel, even in a damaged condition, it may be cost effective to delay minor maintenance and continue monitoring that particular structure.

7-32. Performance and Evaluation

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 postconstruction 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. Modifications to a dike length or elevation can be made if it is found that such is needed. A slightly underdesigned structure or series of structures will probably be more cost effective thau 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.

CHAPTER 8

CANALIZED WATERWAYS

Section I. Principal Features

8-1. Locks and Dams. Some streams can be developed for navigation only through the use of a series of locks and dams, with or without training and channel stabilization structures. Generally, locks and dams would be required in streams having steep gradients with velocities too high for navigation, of inadequate depths particularly during low-water periods, or where conditions make it impractical to develop the required depths by contracting structures because of rock outcrops, sediment movement, or the effects of such structures on velocities affecting navigation and on the flood-carrying capacity of the stream.

8-2. General Description. Lock and dam structures provided primarily for navigation usually consist of one or more locks, a dam or spillway for the maintenance of a minimum upper pool level, and other accessories as required to assist navigation and to provide for the passage of flood flows. These structures are usually of the low-lift type with lifts varying from a few feet to in excess of 30 feet. The lock or locks usually include guide and/or guard walls, an esplanade, and filling and emptying systems. The dam could be of the navigable or nonnavigable type and could include a gated or controlled spillway, overflow weirs and embankments, navigable passes, and overbank embankments.

8-3. Navigable Dams. Navigable dams have been used on some streams with low-lift locks and are provided with a controllable section such as bear traps and a navigable pass which is opened to navigation when discharges are sufficient to provide adequate depths without the effects of the dam. Navigable passes are designed to provide safe transit for all traffic expected to use the waterway when flow conditions permit. Because of the operational difficulties and the frequent inundation of the low-lift locks, these structures are considered more or less obsolete. The navigable-type dams on the Ohio River are being replaced with nonnavigable dams to provide for higher and longer pools, larger locks, and more efficient operation. Navigable dams usually have little effect on flood stages since the elements of the dam are on the channel bed during high water except for small piers and the locks are overtopped. Planning of navigable dams is discussed in EM 1110-2-2606.

8-4. Nonnavigable Dams. Practically all of the newer dams are of the nonnavigable type and are designed to maintain a constant upper pool as long as conditions will permit and to pass the higher flows with a small backwater effect. The magnitude of backwater for nonnavigable dams is determined through a cost effectiveness analysis. This analysis considers several spillway lengths and overflow weirs with the smaller lengths causing a higher upstream backwater elevation. The minimum cost layout will be the most cost effective. With nonnavigable dams the minimum number of gates should be based on operational flexibility, particularly when one or two gates are down for maintenance or repairs. The minimum number of gates should be determined project by project based on experience and engineering judgment. A higher upstream water-surface profile could increase project cost by:

a. Requiring more land acquisition or flood easements.

b. Consequential damages for increased flood heights and groundwater elevations.

c. Requiring longer, higher levees.

d. Requiring higher lock and dam.

e. Requiring more relocation.

The potential savings are reduced size of spillway, overflow weirs, stilling basin, and exit channel widths. Also, a narrower spillway exit channel will keep sediment moving in alluvial streams and reduce maintenance dredging. In some cases, locks and dams are designed for hinged pool operation that would permit the lowering of the pool level some 3 to 5 feet in anticipation of a flood upstream, provide for powerhouse releases upstream, or change the movement of sediment and location of shoal areas. To provide adequate flow capacity within the streambanks and for the passage of sediment, the crest of the sills of the spillway gates is usually at or near the elevation of the natural streambed. When the length of the spillway is less than the width of the channel between the river-side lock and the far bank, an overflow section or weir might be placed within the channel to supplement any overflow structures along the overbank.

Section II. Overflow Sections

8-5. Substitute for Gate Bays. Overflow sections or weirs can also be used as a substitute for some of the gate bays by being placed in a part of the channel cross section usually on the side opposite the lock or locks and are designed to pass with the overbank section some of the flow during higher river stages. Overflow weirs or sections are usually cheaper than the portion of the spillway and stilling basin they replace but also could reduce the maximum discharge at which the normal upper pool level could be maintained. Use of overflow sections to reduce the length of the gated spillway would tend to increase velocities in the lock approaches. The crest of overflow weirs or sections is generally 2 to 3 feet above the normal upper pool elevation.

8-6. Design for Navigation. Overflow sections are sometimes designed to provide for navigation over the structure during high water; this would permit the lowering of the top of the lock walls. In some cases, short navigable passes consisting of overflow sections are installed in nonoverflow embankments for the passage of emergency plant and equipment when the locks are out of operation because of high water. The adverse effects of frequent inundation of the locks because of the preparations required, the cleanup before the locks can be placed back in operation, and the difficulty in predicting river stages are factors to be considered in deciding the amount to lower the tops of the lock walls. The navigability of overflow sections depends on the length of the section, the head over the section, and the alignment of currents upstream and downstream. Studies have indicated that upbound tows can negotiate the overflow section with a head up to 0.8 foot or more, depending on the power of the boat and load. Tows with limited power might encounter some difficulty in negotiating heads greater than about 0.5 foot if the section is approached at a slow speed. Generally, tows maintaining sufficient momentum to move the lead barges across the weir would encounter little or no difficulty if power is adequate to navigate against some of the higher velocity currents encountered in the stream. With the overflow section positioned next to the gated spillway and flow confined on the landward side, an upbound tow could block a sizable portion of the flow over the weir, causing an increase in water level on the land side of the tow. If the overflow section is relatively narrow, the difference in water level on the side of the tow could be sufficient to move the tow toward the spillway, causing the tow to slam against the abutment pier. The same condition could occur with the tow moving across the weir at a sizable angle to the direction of flow.

Section III. Effect of Structures on Currents

8-7. Upper Lock Approach. Locks placed in the channel of a stream form an obstruction to a portion of the flow of that stream. The effects of these structures on currents depend principally on the configuration and alignment of the channel upstream and downstream therefrom and the amount of contraction and expansion in channel width produced by the obstruction.

The usual effect of the sudden channel contraction in the upper approach to the locks is an outdraft or crosscurrent that affects the movement of a tow at a time when its rudder power is reduced because of reduced speed with respect to river currents. The intensity of the crosscurrents is dependent on the total discharge affected by the structure and is a function of the velocity of currents approaching the structure, channel depth, and width of channel affected by the structure, and, in some cases, by flow along the adjacent overbank. Since no two reaches of a stream are identical, the intensity of the crosscurrents in the upper lock approach will vary according to the site selected and the orientation of the structures with respect to the alignment of the channel and currents.

8-8. Lower Lock Approach. Because of the sudden expansion in channel width downstream of the lock or locks, a tendency for an eddy to form in the lower lock approach will exist. The eddy produces currents moving landward at its downstream end, upstream currents along its landward side, and currents moving riverward at its upstream end. A tow moving toward the lock with little or no rudder power because of reduced speed and upstream currents is affected by these currents which are constantly varying in size and intensity. Currents in the lower approach can also be affected by lock emptying, powerhouse releases, uneven gate operation, flow from or toward the overbank, and flow from tributary streams. Unless locks are carefully designed, these effects could seriously affect the movement of tows in the lock approach. Since conditions vary at each site and cannot be fully resolved by analytical means, hydraulic model studies with model towboat and tow are usually required to assure safe and efficient passage through the lock or locks.

Section IV. Lock Auxiliary Walls

8-9. Guide Walls. Guide walls are used to assist tows in becoming aligned for entrance into the lock chamber without jamming of the lock gates when the gates are recessed in open position. Guide walls for single locks are usually on the land side and have all, or at least a sizable portion, of their length straight with their lock-side face in line with the inside face of the adjacent lock wall. Guide walls themselves provide tows little or no protection from the currents, but mooring lines can be attached to the wall to assist tows in overcoming the effects of adverse currents. When currents are not a factor such as in a canal or lake the guide wall is usually placed to provide the best protection from the prevailing winds. Short guide walls angled away from the approach channel are generally provided on the opposite wall to prevent tows from hitting the end of that wall.

a. Upper Guide Wall. Currents along the upper guide wall force downbound tows approaching the wall to move close to the wall; mooring lines are attached that are used for snubbing the tows into alignment. If the pilot misjudges the currents, the tow is in danger of either hitting the end of the wall or moving too far from the wall to attach mooring lines. Tows angling toward the guide wall to attach mooring lines are in danger of having their stern moved riverward, toward the spillway, because of the decrease in rudder control created by the necessity to reduce speed when approaching the wall. Usually, the wall can be approached safely by cautiously flanking toward the wall, attaching mooring lines, and snubbing the tow into alignment after mooring lines are attached. Upbound tows leaving the locks could also be affected by outdraft or crosscurrents that would tend to move the head of the tow riverward before the entire tow cleared the lock chamber. Because of the danger mentioned and delays that could be experienced in maneuvering for the approach, an upper guide wall without a guard wall or other protection is not recommended for single locks where currents of sizable magnitude can be expected along the wall and in the lock approach.

b. Lower Guide Wall. Upbound tows approaching the lower guide wall for entrance into the lock would encounter eddy currents which vary in size and intensity. Tows approaching the wall could encounter upstream currents along the wall and riverward currents at the upper end of the wall. Here again if the pilot misjudges the strength and position of the eddy at the head of the tow, he is in danger of hitting the wall or of passing too far from the wall to attach mooring lines. Experience indicates that eddy currents exceeding 1 foot per second are objectionable, and even currents of lower velocity could be a nuisance since they tend to move tows away from the wall. This tendency can be overcome by increasing power on the towboat or by attaching a mooring line to the wall. Conditions created by a lower guide wall are generally not as hazardous as conditions in the upper approach; nevertheless, they could cause considerable delays, depending on the intensity of the eddy and experience of the pilot. In some cases, the objectionable condition can be minimized or even eliminated by installing low structures along the river side of the approach channel.

8-10. Guard Walls. Guard walls provide tows with some protection from adverse currents and are usually on the spillway side of the lock. Guard walls might be used in addition to the regular guide wall or could be designed to serve as a guide wall also. Guard walls may be solid, ported, or spaced intermittently, depending on their purpose and their alignment relative to that of the lock and currents.

a. Upper Guard Wall. The upper guard wall can be an important factor in the safety of downbound tows and protection of the structures. The upper guard wall, when used as a guide wall, is generally as long as the clear portion of the lock chamber. Once the tow is behind the wall, it is safe from the effects of currents that would otherwise move the tow toward the spillway. With a solid upper guard wall, crosscurrents near the end of the wall would tend to move the head of downbound tows riverward and put them in danger of hitting the end of the wall. Also, there would be a tendency for an eddy to form between the wall and the adjacent bank, producing a riverward current near the upstream end of the wall and a landward current some distance downstream. Downbound tows must reduce speed as they approach the end of the wall, thus losing steerageway and the ability to overcome the effects of these currents. The danger involved depends on the intensity of the currents and the distance between the wall and adjacent bank. The landward currents near the downstream end of the eddy are usually not serious; however, they could slowly move the head of a stopped downbound tow away from the wall. The intensity of the crosscurrents depends on the amount of flow the guard wall tends to intercept. Upper guard walls are generally straight, especially when the lock is adjacent to the spillway. Flaring of the guard wall would increase the amount of flow the wall intercepts and could affect the distribution of flow through the spillway gatebays near the lock. Crosscurrents near the end of the guard wall can be eliminated or their effects minimized with properly designed ports in the wall. Design of ports in guard walls is discussed in a subsequent paragraph. Crosscurrents near the end of the wall could also affect upbound tows leaving the lock, moving their heads riverward before they had cleared the wall.

b. Lower Guard Wall. Lower guard walls provide tows protection from currents resulting from spillway discharge, uneven gate operation, powerhouse releases, and lock-emptying outlets located on the river side of the lock. Generally, an eddy will tend to form in the lower lock approach downstream of the end of the wall. The currents in the eddy move toward the adjacent bank at its lower end, then upstream along the bank, and riverward on its upstream end, just downstream of the end of the guard wall. The eddy will tend to move the head of an upbound tow riverward as it approaches the end of the wall. With a guard wall, this condition is not serious since the tow can approach the end of the wall some distance landward of the wall and the outdraft will assist the tow in moving toward the wall. When a tow is stopped with a portion of the tow extending beyond the wall, the currents would tend to move the stern riverward and could cause the head of the tow to move away from the wall. This movement could be resisted with some power on the towboat or mooring lines attached to the wall. The lower guard wall when used as a guide wall is usually the same length as that of the lock chamber; however, under some conditions it could be one half to two thirds that length depending on the alignment and intensity of currents.

Section V. Arrangement of Locks and Auxiliary Walls

8-11. Single Lock. Walls used to assist tows in approaching and entering the locks vary in type and arrangement. In their simplest form, single locks might include guide walls, guard walls, or a combination of both. Guide walls are usually on the land side of the lock (fig. 8-1a); guard walls are usually on the river side (fig. 8-1b). Some locks have an upper guard wall and a lower guide wall (fig. 8-1c). The upper gate pintles of most locks are along the axis of the dam so as to place the lock chamber in the lower pool, reducing pressure on the lock walls when lock is dewatered. When the guard wall is of sufficient length, it also serves as a guide wall for the lock.

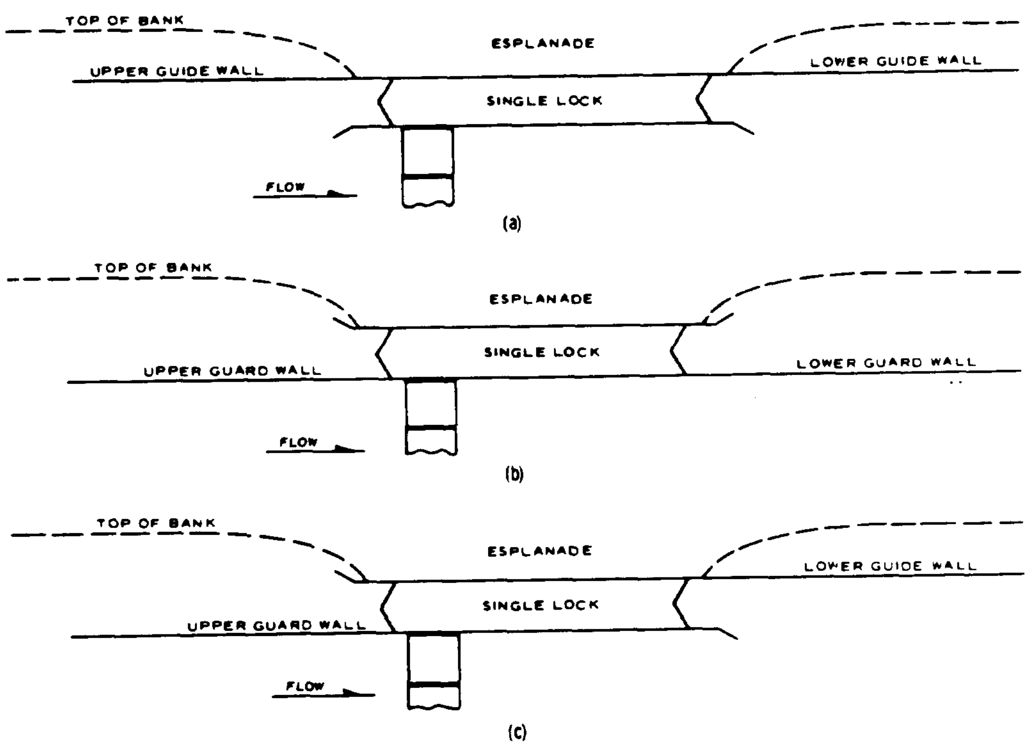


Figure 8-1. Arrangement for single lock

8-12. Adjacent Locks. With two adjacent locks, there is a common inter-mediate wall. The general practice has been to equip the river-side (main) lock with a ported upper guard wall and a solid lower guard wall. The land-side (auxiliary) lock usually has an upper guide wall (land side of the lock) ad a lower guide wall. When the land-side lock is shorter than the river-side lock, the land-side face of the intermediate wall extending beyond the end of the auxiliary lock could be used as the guide wall (fig. 8-2a). When the upper guard wall is ported, tows tend to be moved toward the guard wall because of flow through the ports, making it somewhat difficult for downbound tows to approach the guide wall for passage through the land-side lock. The difficulty is increased when the land-side lock is the same size as the river-side lock. Another problem confronted with this arrangement is that a tow cannot safely approach either lock when another tow is leaving or tied up along the guide or guard wall, resulting in delay for approaching or departing tows.

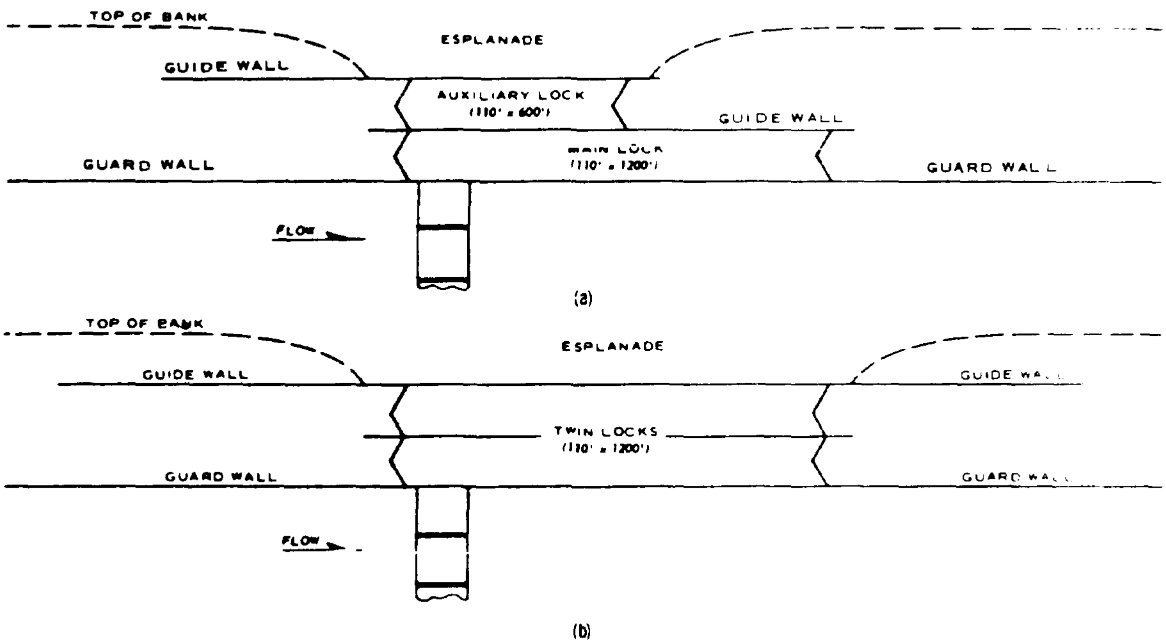
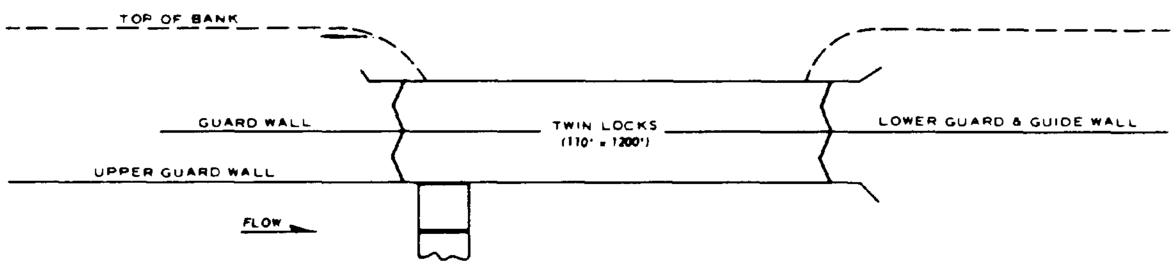


Figure 8-2. Arrangement with adjacent locks

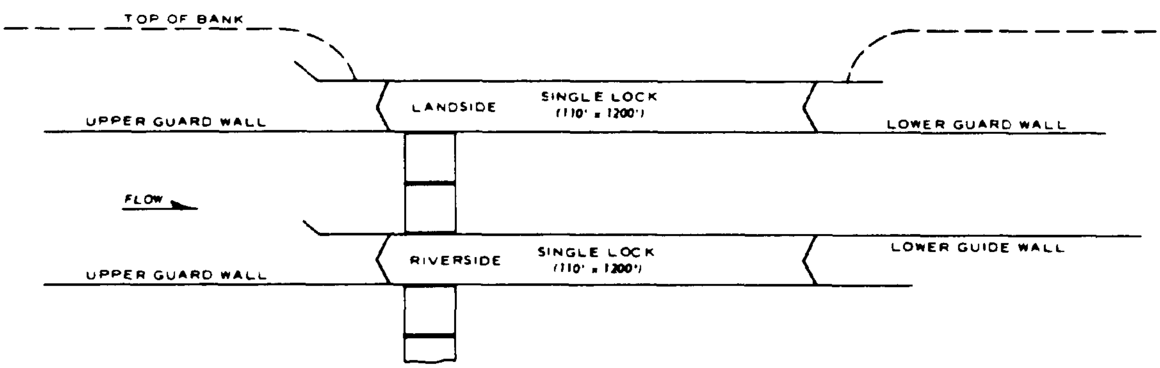
Section VI. New Arrangements of Locks and Auxiliary Walls

8-13. General. Based on the results of model studies, new concepts in lock arrangements have been developed to provide safer and more effi-cient movement of traffic through the locks. These concepts were developed for use with two parallel locks of the same size. The first concept involves the use of adjacent locks; the second involves the use of separate locks.

8-14. Upper Lock Walls with Adjacent Locks. The new concept is to provide an upper guard wall for both locks when the locks are adjacent (fig. 8-3a). Both guard walls would have to be ported. The land-side guard wall should be at least half the length of the usable portion of the lock chamber and the river-side guard wall should be of sufficient length to extend at least three fourths of the length of the usable portion of the lock chamber beyond the end of the guard wall for the land-side lock. These lengths are based on limited tests with specific projects and some variations might be desirable, depending on local conditions. The same arrangement could be used with adjacent locks of different sizes with the upper gate pintles of both locks along the axis of the dam. Since there would generally be little flow through the ports in the land-side lock guard wall, the tops of the ports should be a few feet higher than those in the river-side wall to develop



(a)



(b)

Figure 8-3. New concepts in lock arrangement currents that would assist tows in approaching the wall. As a result of this arrangement, a downbound tow could approach the river-side lock and be followed by a downbound tow approaching the land-side lock as soon as the tow using the river-side lock has landed along the guard wall. Also, a downbound tow using the land-side lock can approach the lock while an upbound tow is leaving the river-side lock; and a downbound tow could approach the river-side lock while an upbound tow is leaving the land-side lock, provided the head of the upbound tow does not extend beyond the end of the land-side guard wall.

8-15. Lower Lock Walls with Adjacent Locks. In the lower approach, the new arrangement provides for the extension of the intermediate wall to form a guide wall for both locks (fig. 8-3a). The land-side and river-side faces of the wall would have to be in line with the inside faces of the adjacent locks and constructed to withstand the impact of tows approaching the wall from either side. With this arrangement, tows entering or leaving one lock would not interfere with tows entering or leaving the other lock. For safe two-way traffic, the length of this wall should be the same as that of the tows using the locks. In addition to the advantage of two-way traffic, a long intermediate wall would cause a more gradual increase in channel width than with a long river-side guard wall, thereby reducing the shoaling tendency and, in turn, maintenance cost and interference with traffic during maintenance dredging. Shoaling, if any, would start in the approach to the river-side lock where it could be removed without interfering with traffic using the land-side lock. The only disadvantage attributed to this scheme is that upbound tows approaching the river-side lock would use more power since they would be moving farther out into the channel in higher velocity currents and would encounter some currents along the river side of the wall. However, this disadvantage is more than offset by the elimination of delays and power required to maintain position in the stream while waiting for other tows to clear the locks.

8-16. Separation of Locks. The second concept involves separation of the locks to provide two-way traffic in either or both directions (fig. 8-3b). The amount of separation required is presently a matter of opinion and could vary depending on local conditions. Navigation interests have indicated that a separation of about 270 feet was acceptable in the case of a replacement structure on the upper Mississippi River. Based on the movement and passing of tows through restricted reaches and bridge spans and on results of model studies, it appears that separations of about 200 feet or less might be adequate under most conditions. Separation of the locks would produce a greater obstruction to flow and result in an increase in crosscurrents in the lock approaches. To reduce the effects of the obstruction, spillway gates should be provided between the locks to pass some of the flow affected by the locks. This would reduce the crosscurrents produced by the total flow moving toward the spillway across the riverward lock approach; the size and intensity of the eddy that would be developed in the lower approach would also be reduced since the amount of channel expansion would be reduced. The hydraulics involved in the development of satisfactory navigation conditions with lock separation are more complex than for adjacent locks; design should not be finalized without benefit of a model study.

a. Upper Approach. In the upper approach to the locks a guard wall would be required on each lock. The guard wall would be on the river side of the land-side lock and could be on either side of the river-side lock depending on flow conditions and configuration of the channel upstream of the lock. The lengths of the upper guard walls should be at least three fourths the length of the usable lock, depending on the currents existing after completion of the project. The river-side lock upper guard wall generally needs to be longer than that of the land-side lock and, in most cases, at least as long as the lock chamber.

b. Lower Approach. In the lower approach the guide wall could be on either side of each lock. In most cases, it would be better to have the guide wall on the river side for the landward lock and on the land side for the riverward lock. This would provide greater separation of traffic approaching and leaving the lower lock approach. The length of the guide wall on the land-side lock should be at least half the length of the usable lock chamber and on the river-side lock at least two thirds of the length of the lock chamber.

8-17. Lock on Each Side of Channel. Lock separation could also be accomplished by placing locks on both sides of the channel with the gated spillway between the locks. This arrangement would be ideal for two-way traffic and would be preferred by navigation interests. In most streams it would be extremely difficult to develop currents in both lock approaches that would not be objectionable to navigation, particularly during higher flows. However, in an alluvial stream where the movement of sediment is involved, it would be impractical since the development of a channel of adequate depth on both sides of the river would require considerable maintenance dredging. Training structures designed to provide adequate depths would tend to affect navigation conditions in the lock approaches and could affect flow through the spillway. This arrangement could be practical in some straight reaches of channels carrying little or no sediment. The need for additional operating personnel would increase the cost of operation with this arrangement.

8-18. Locks in Canal. Locks located in a canal bypassing the dam in the main channel should be provided with guide walls to assist tows in becoming aligned for entrance into the lock. Since there are usually little or no currents in the canal, walls can be shorter than those required in the main channel, particularly with a single lock. When twin locks are located in the canal, extension of the intermediate lock wall can be used as guide walls in the upper and lower approach as shown in figure 8-4. This arrangement should be less costly than separate guide walls for each lock and could permit two-way traffic under most conditions because of the separation provided by the center wall.

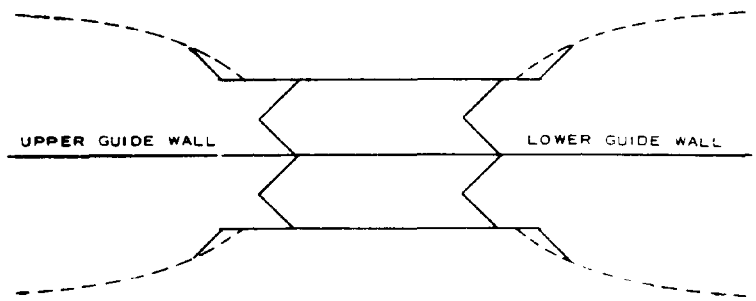


Figure 8-4. Twin locks with common guide walls

Section VII. Lock Size and Number

8-19. Lock Selection. The selection of the size and number of locks should be based on the requirements of the anticipated waterway traffic, with consideration of the characteristics of the waterway on which located. A thorough study of the equipment which will likely be using the lock and the size of barges and tow formations favored by the towing industry should be made before selection of lock sizes. The trend in barge construction has been toward larger units, especially for liquid cargos, varying from 35 to 48 feet in width and 195 to 300 feet in length. The type of bulk commodity that would be moved on the waterway would influence to some degree the size of barges and tow formations. The following are the standard lock sizes recommended by the Corps of Engineers:

Usable Lock Dimensions, Feet	
Width	Length
84	400
84	600
84	720
84	800

(Continued)

Usable Lock Dimensions, Feet	
Width	Length
84	1200
86	675*
110	600
110	800
110	1200

* Columbia and Snake River only.

Deviations from the above might be justified under special conditions and in the interest of safety and efficiency. On waterways where only small size crafts can be accommodated, smaller locks would be adequate. The use of more than one lock would depend on the volume of traffic expected and whether or not closure of the lock for repairs can be afforded.

Section VIII. Special Lock Features

8-20. Filling and Emptying Systems. Time required for tows to pass through a lock is affected by approach conditions, lock filling and emptying time, and to some extent the elevation of the lock floor and sills. **Under maximum head most locks are designed to permit filling and/or emptying in 6 to 12 minutes. The details of the various types** of filling systems that meet the requirements with least cost are covered in EM 1110-2-1604. The types generally recommended are:

Lock Lift, Feet	Filling System
0-10	Front end (sector gate or lock culvert)
10-40	Side ports
Over 40	Horizontal split bottom longitudinal

Sketches of these filling systems are shown in figures 8-5 through 8-7. The effects of lock filling and emptying on navigation conditions are discussed in another section of this manual.

8-21. Chamber Floor and Gate Sill. The lock floor elevation is established to allow rapid filling and emptying while maintaining reasonable

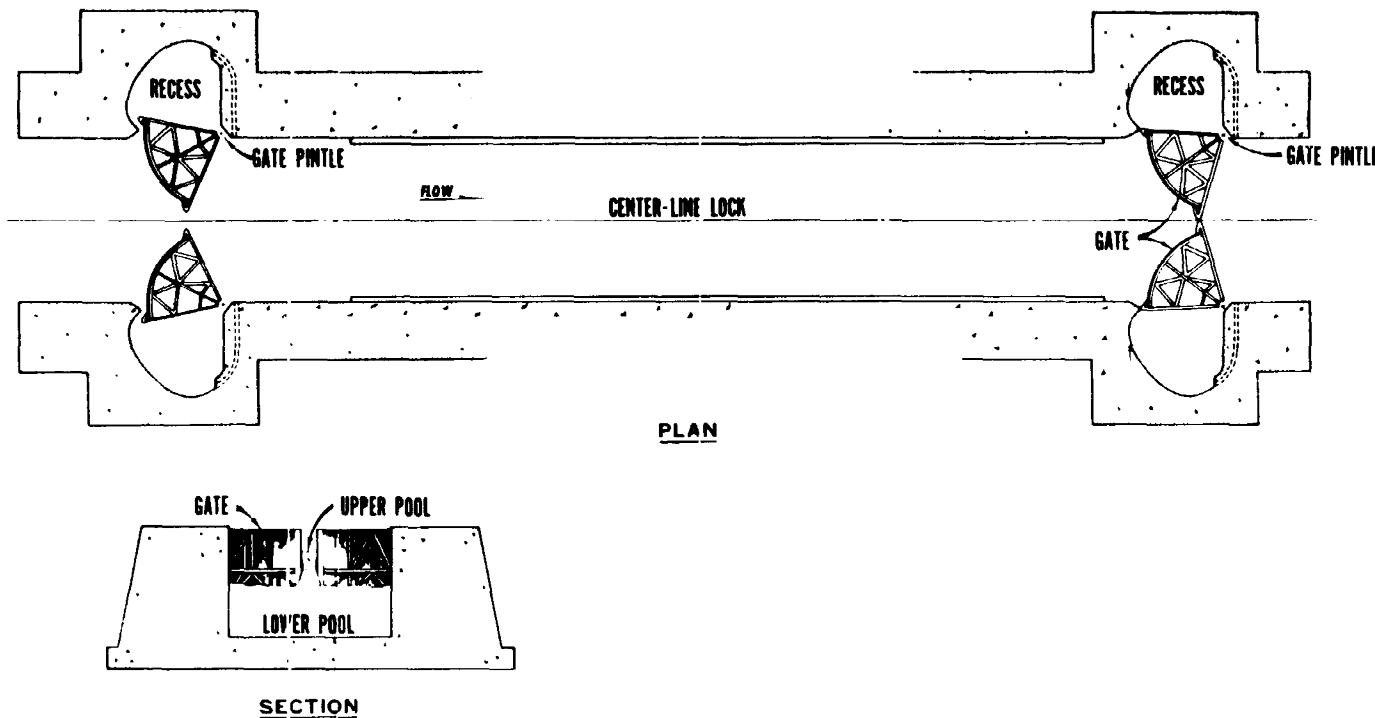


Figure 8-5. Sector gate

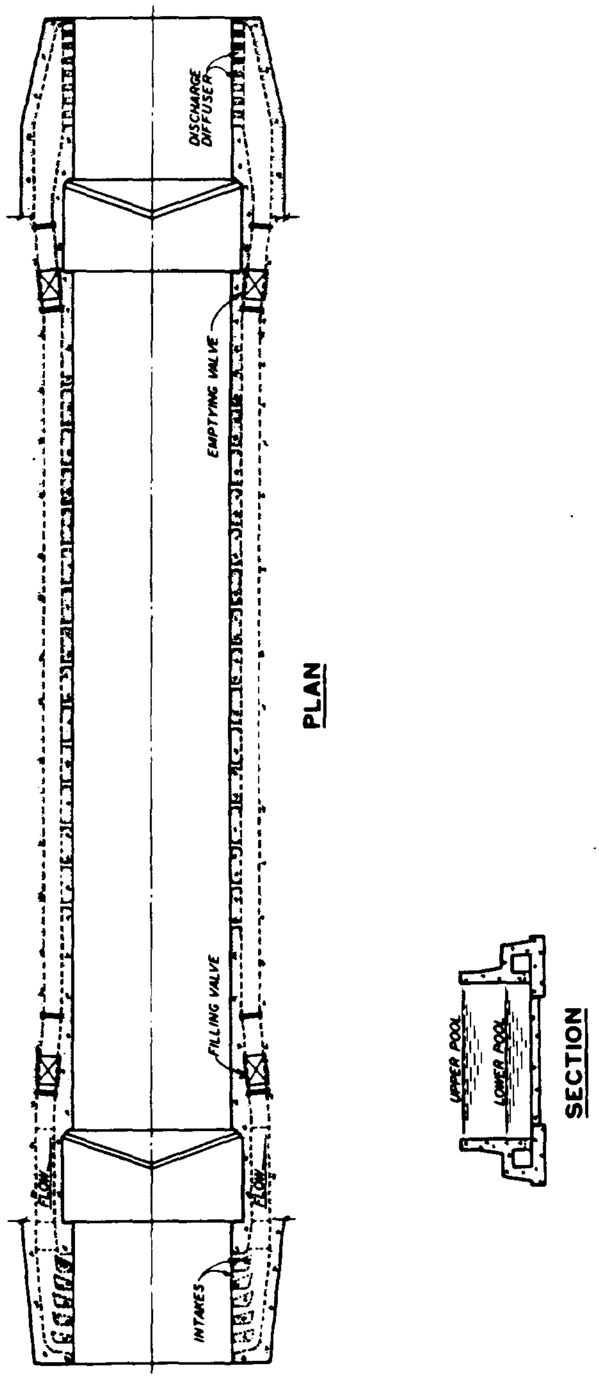


Figure 8-6. Sidewall port

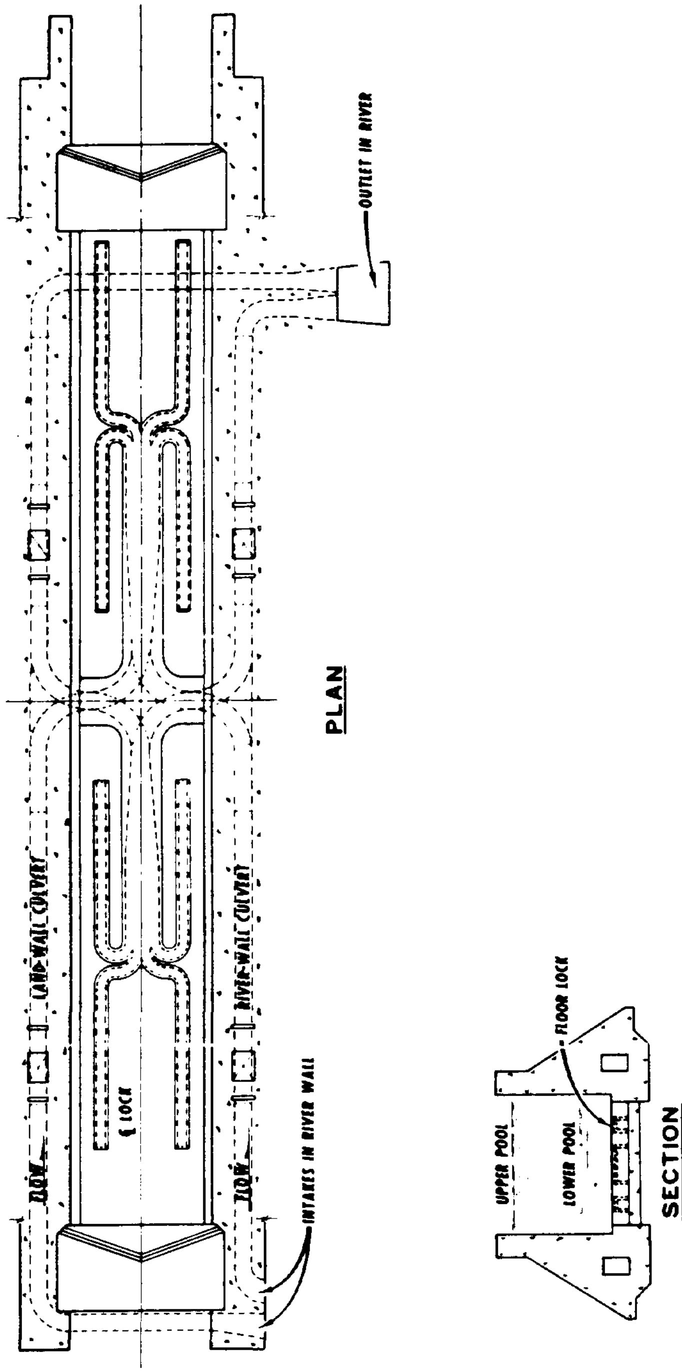


Figure 8-7. Longitudinal floor culvert

hauser loads. To meet this objective, an adequate space must be provided between the vessel bottom and floor so the filling water jets do not strike the vessel. When the lock floor is established the sill elevation can be determined. Detailed studies using model test results will define floor elevation more exactly. The gate sill should be as low as possible to allow a large water cross section for displaced water to exit the chamber (fig. 8-8). A 2- or 3-foot-high sill (above chamber floor) is often desirable to provide a space for gate seating and maintenance work and to keep sediment and debris out of the chamber. The minimum depth over the upper gate sill should be at least the same as the minimum depth over the lower gate sill. Comparison studies show increased cost for a higher gate is about equal to the savings in concrete for a low sill. Sector gated locks (front end filling system) usually do not have sills; the gate bottom is the same elevation as the chamber floor.

8-22. Lock Walls. The height of the lock walls should depend on the importance of the waterway and protection required for navigation, the characteristics of the waterway and type of dam selected, type of lock structure in connection with the foundation available, balance between initial and maintenance cost, need for uninterrupted traffic during high stages, and other conditions that might be peculiar to a given location. On important waterways where commercial traffic is not interrupted by currents, floating debris, ice, or other navigation hazards serious consideration should be given to providing lock walls of sufficient height to accommodate traffic during all but the most infrequent floods. If traffic cannot be accommodated during most of the year or is subjected to frequent interruptions, the full potential of the waterway would never

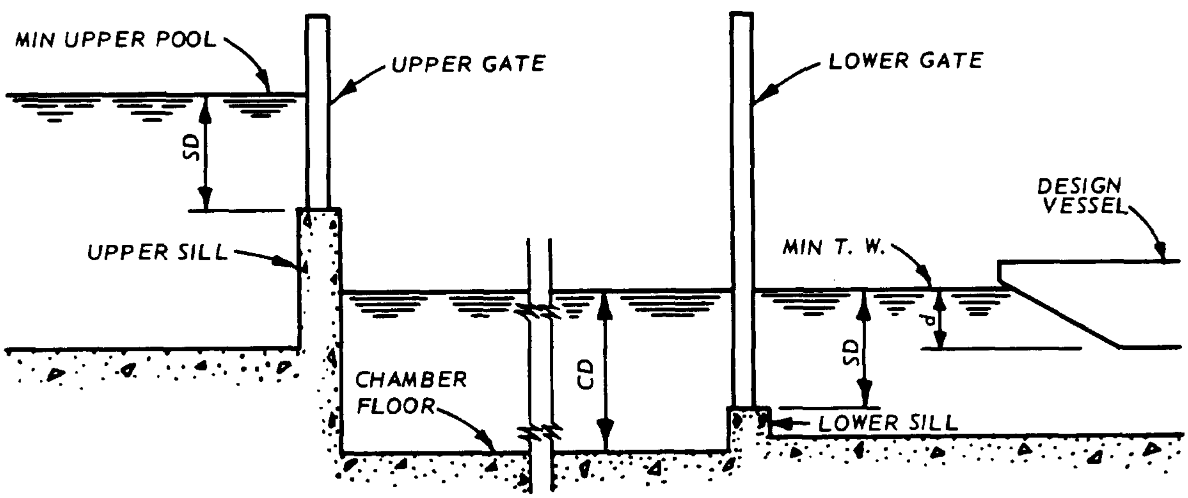


Figure 8-8. Lock longitudinal profile

be realized. Generally, the tops of the lock walls are set so that the longest period of traffic interruption would not exceed 10 to 15 days during the largest flood of record. In order to provide protection for tows, particularly those with empty barges, lock walls should be at least 2 to 3 feet above the maximum navigable stage depending on currents and wind that could affect navigation. In some cases, a considerable saving in initial construction cost can be realized with lower lock walls and a navigable overflow section in the dam with an operative system suitable for frequent submergence. However, frequent submergence can present many problems and increase operation and maintenance cost because of preparations required, cleanup and restoration, and difficulty of predicting changes in river stages.

8-23. Types of Construction. The resistance of concrete to impact, abrasion, and deterioration has caused this type of construction to be accepted as the most suitable for Corps projects. This type of structure permits the use of fast filling and emptying systems and should have a life expectancy in excess of 50 years. Sheet-pile locks have been constructed on the Ohio River (Locks 52 and 53) to alleviate congestion until permanent structures can be provided. These locks have a relatively low initial cost but generally have longer filling and emptying time, high maintenance cost, and a life expectancy of only 15 to 25 years.

CHAPTER 9

SITING OF LOCKS AND DAMS

Section I. Factors Involved

9-1. General. Locks and dams are usually placed within the channel cross section of streams or rivers with or without modification of the channel. In some cases, locks and dams are placed within a cutoff channel or the locks might be in a canal with the dam or spillway located separately in the main channel. Regardless of the layout used, special navigation problems could be encountered that should be anticipated and resolved before the final design is adopted.

9-2. Locks in Stream Channels. Navigation conditions in the approaches to locks placed in a flowing stream will depend largely on the alignment of the channel and channel configuration upstream and downstream. It is important that currents approaching the lock be slow to moderate and reasonably straight within the approach for a considerable distance upstream. Generally, the lock should be sited where downbound tows can complete any change in direction and become properly aligned for the approach before having to reduce speed. Also there should be sufficient sight distance to preclude the danger of collision or interference with other traffic and to permit the tow to maneuver as required for the approach. These requirements indicate the need for locks to be located in reasonably straight reaches. Because of the characteristics of natural streams and other considerations such as foundation conditions, flowage easement, etc., ideal conditions are seldom, if ever, available.

9-3. Other Considerations. The site selected for the lock and dam structure should be one of the most important factors in the development of satisfactory navigation conditions. In addition to other factors, the design engineer should consider existing conditions in the upstream and downstream reaches of the proposed sites (including current directions and velocities), sediment movement for the various flows possible, effects of the structures on the currents and movement of sediment, effects of the resulting currents on the movement of tows, and foundation conditions. The characteristics of the foundation material determined during the early stages of the investigation should provide some indication of the probability that the structures needed can be constructed at reasonable cost with ordinary design standards and could reduce the number of sites considered.

Section II. Channel Alignment

9-4. Effects of Channel Alignment. Locks are usually located along one bank adjacent to one end of the dam. Natural streams having erodible bed and banks will tend to develop a sinuous course consisting of alternate bends and crossings with some relatively straight reaches. The alignment of the channel upstream and downstream of the proposed site will affect visibility and currents that influence the movement of tows approaching the lock. As a general rule, locks and dams should not be located in a bend unless it is a relatively long flat bend.

9-5. Locks on Concave Side of Bends. Locating locks within a bend on the concave side would facilitate the development and maintenance of navigable depths within the lock approaches; however, conditions would be affected by the heavy concentration of flow and high-velocity currents on the lock side of the channel. This condition is aggravated in relatively short-radius bends where the locks have to be placed some distance from the bank to provide adequate sight and approach distance. Usually the best location for navigation in a natural channel is a straight reach downstream of a bend. With locks located on the bank forming a tangent to the concave side of the bend, tows would not have to make a crossing or turn before approaching the lock; thus currents would tend to keep the tow on that side of the river.

9-6. Locks on Convex Side of Bends. Locks located on the convex side of a bend would affect less of the total river flow but would require downbound tows to make a turn for the approach which would place the stern riverward of the bank line in currents moving toward the spillway. Also, there would be a tendency for shoaling on the convex side of the channel. Since many accidents and delays have been experienced by downbound tows attempting to approach locks on the convex side of the bend (such as with Gallipolis Locks and Dam on the Ohio River and Locks and Dam 26 on the Mississippi River), this site should be avoided if at all practical. If such a location cannot be avoided, the locks should be placed far enough downstream to permit downbound tows to negotiate the turn and become aligned for the approach before having to reduce speed and lose rudder control.

9-7. Bypass Canals. Where short-radius bends cannot be avoided, consideration should be given to the construction of the lock or locks and dam in a cutoff channel across the bend. Such location would require considerable excavation but would reduce cofferdam requirements since some of the structures could be constructed in the dry before excavation is completed. With the lock located in a bypass canal and the dam in the existing channel, careful consideration must be given to entrance and exit conditions at each end of the canal.

9-8. Factors to be Considered. Before the final selection of a site within the existing channel is made, information regarding channel depths and alignment, overbank elevation, and current direction and velocity for all of the navigable flows should be gathered and analyzed with regard to conditions that might result from construction of the structures. Previous studies indicate adequate data are seldom, if ever, available to permit a reasonable analysis of the conditions existing in the reach considered. Time usually will not permit an adequate survey of the reach, particularly since some of the flows that should be considered might not be experienced for several years. In cases where data are limited or the effects of the structures on navigation conditions cannot be fully resolved analytically, use of model studies is highly recommended. These studies would be used to determine the adequacy of the proposed site, the best arrangement and alignment for the structures, and any modifications that might be needed to eliminate undesirable conditions.

Section III. Locks in Canals

9-9. Effects on Navigation. Locks and spillway portions of dams placed in a canal or cutoff channel are subject to the same lock approach conditions as those that would prevail with the structures in a natural channel of the same general alignment. With the lock in a canal bypassing the spillway and dam, navigation conditions could be affected by currents across the upper and lower entrances to the canal, by flow across the canal toward and away from the spillway during higher flows, and by flows caused by lock filling and emptying, depending on the location of the intake ports and emptying outlet. Conditions at the upper entrance to the canal can be extremely hazardous, particularly for downbound tows, because of currents moving across the entrance toward the spillway. When the head of a downbound tow enters slack water, the currents tend to rotate the stern of the tow downstream. If the tow is reducing speed because of a narrow entrance or presence of other traffic, it is in danger of hitting the river-side bank or is in a position to hit the opposite bank.

9-10. Upper Canal Entrance. Flaring of the canal bank on the river side to increase the size of the opening would increase the flow moving across the entrance, producing undesirable conditions for downbound tows entering the canal. It is better to maintain a straight bank along the spillway or river side of the entrance and flare the land side as far as conditions will permit.

9-11. Two-Way Traffic. For two-way traffic, downbound tows should enter the canal from along the land side and upbound tows pass on the river side. With the upper entrance to the canal just downstream of a bend on the concave side, velocities on the canal side will tend to be high, increasing the intensity of the currents moving across the entrance to the canal.

9-12. Flow Across Adjacent Overbanks. Land between the canal and stream should be high enough to prevent any appreciable flow across or from the canal toward the spillway channel during all navigable flows; if it is not, a fill or dike should be placed along the stream side of the canal. When a dike or fill is placed on the river side of the canal, flow from the overbank on the land side of the canal could also increase the flow across the entrance to the canal. If such is the case, flow along the overbank should be diverted riverward by a dike along the overbank some distance upstream of the entrance.

9-13. Lock Filling. Filling the lock from the canal could produce surges varying from a few tenths of a foot to several feet in water-surface elevation peak-to-trough which could adversely affect navigation and operation of the lock. The magnitude of the surge would depend on the length, width, and depth of the canal and the rate and frequency of lock filling. Surges could cause barges to hit the bottom of the canal during the trough of the surge wave if adequate depths are not provided to compensate for its effects. Currents varying in intensity and direction which cannot always be anticipated by the pilot would also develop within the canal. The change in the water-surface elevation caused by the surge would also affect the head on the upper lock gate and could cause delays in opening of the gate. Surges in a canal can continue for several hours and if successive lock fillings occur, the magnitude of the surge can be several times greater than that for a single lock filling. Filling of the lock from the river side of the canal would eliminate the tendency for surges; however, a difference could result between the water levels inside the lock and in the canal at the end of the lock-filling operation that might require a special auxiliary filling system or a special gate-opening mechanism.

9-14. Reduction of Surges in Canal. The magnitude of surge in a canal can be reduced by reducing the length of the canal approaching the lock; increasing the cross-sectional area of the canal, particularly depth; using a surge basin near the lock-filling intake; and permitting some riverflow through the canal by providing a ported guard wall on the lock with outlet discharging into the river channel upstream of the spillway. With ports in the guard wall on the river side of the lock, there would be some flow into the canal that would reduce the intensity of the crosscurrents near the canal entrance and facilitate the entrance of downbound tows into the canal. Flow through the ports in the upper guard wall would also produce currents that would assist tows in moving toward the wall and becoming aligned for entrance into the lock. The tops of these ports would have to be below the depth of the bottom of loaded barges to prevent the tow from being held against the wall.

9-15. Lower Canal Entrance. Conditions at the lower entrance to the canal can also be affected by currents moving across the entrance and lock approach and by surges created by lock-emptying when the emptying outlet is in the canal. Navigation conditions in the lower reach of the canal (downstream of the lock) would depend on the location of the entrance to the canal relative to currents in the main channel. The entrance to the canal should be aligned as nearly parallel to the alignment of the currents as conditions will permit and flared on its landward side. Flow across the canal should be prevented by high ground or a dike installed along its river side. In streams carrying sediment, shoaling in the lower entrance to the canal could be a serious problem and should be considered in the design of the project.

Please note that Chapter 10 has been replaced with new guidance:

http://www.usace.army.mil/publications/eng-tech-ltrs/etl1110-2-562/toc.html

CHAPTER 11

PLANNING FOR CONSTRUCTION

Section I. Construction Requirements

11-1. Factors to be Considered. The selection of a site for and the arrangement of the lock and dam structures require consideration of problems likely to occur during construction. The effects of the cofferdam on flood stages, the need for passing traffic (if the stream is presently a navigable waterway), and the amount of protection and maintenance required are important factors that could affect the cost of the project. During the construction of nonnavigable type dams, it will be necessary to construct at least one lock before the river is blocked to open-river navigation to maintain navigation during construction. Conditions in the lock approaches with the final-stage cofferdam under construction will be different from those with the cofferdam completed and in place.

11-2. Maintenance of Traffic. Where traffic is to be maintained during construction of the final cofferdam phase, the upper lock gate sill and upper lock approach channel should be low enough to pass traffic during the low flows. Where a guard wall with ports is provided, some arrangement should be made for at least partial closure of the ports to prevent tows from becoming pinned against the wall and to protect small boats when the water level is below the ultimate normal pool elevation. The closures usually consist of curtains constructed of metal, concrete, or other suitable material extending from the top of the ports down, but not necessarily to the bottom of the ports. During partial closure of the ports, the tendency for bed scouring at the bottom of the ports will be increased. Closure of the ports by curtains will increase the tendency for crosscurrents near the end of the guard wall and could affect tows entering or leaving the lock, particularly during the higher flows when open-river conditions prevail. When the final-stage cofferdam is adjacent to the lock, flow from the completed portion of the dam could cause currents to be directed toward the lower guard wall, producing scour along the wall and strong eddy currents in the lower lock approach. Conditions for navigation through the lock would be better, and in most cases, there would be less danger of affecting the stability of the structure with the last cofferdam stage on the opposite side of the channel.

Section II. Cofferdam Design

11-3. Effects on River Currents. Cofferdams obstructing partial riverflow will tend to cause scour, particularly near the upstream corner on the river side. The depth of scour, which could be appreciable, depends on the amount of flow affected by the cofferdam, shape of the cofferdam, and the erodibility of the channel bed. Cofferdams having square corners on their upstream side would tend to scour deeper than those with rounded corners or those with upper arms angled less than 90 degrees to the direction of flow.

11-4. Cofferdam Configuration. The scour along the riverward face of the cofferdam can be minimized by the use of a deflector. Rounded corners or deflectors designed to streamline flow will tend to reduce the depth of maximum scour but would maintain high velocities along the riverward face of the cofferdam. Deflectors can be designed to reduce or eliminate the high velocities along the main part of the cofferdam. Deflectors consisting of an upstream extension of the riverward arm of the cofferdam with a section angled about 45 degrees landward have been successful in containing the scour near the corner of the deflector and along the deflector itself, away from the main part of the cofferdam under pressure when dewatered (fig. 11-1). The length of the extension and the angled portion of the deflector would be based on the amount of contraction provided by the cofferdam and velocities of riverflow. The use of 150- to 200-foot upstream extensions with deflector arms at least that length has produced satisfactory results in tests of Mississippi and Ohio Rivers projects when the river channel was contracted as much as 50 percent. This type of deflector caused deposition along the riverward face of the cofferdam and moved downbound tows away from the cofferdam (fig. 11-2). The downstream arm of the cofferdam, extending normal to or at an angle of not more than about 45 degrees in relation to the direction of flow, would generally be subjected to little or no scour since sediment moved along the riverward arm would tend to be deposited downstream of the cofferdam.

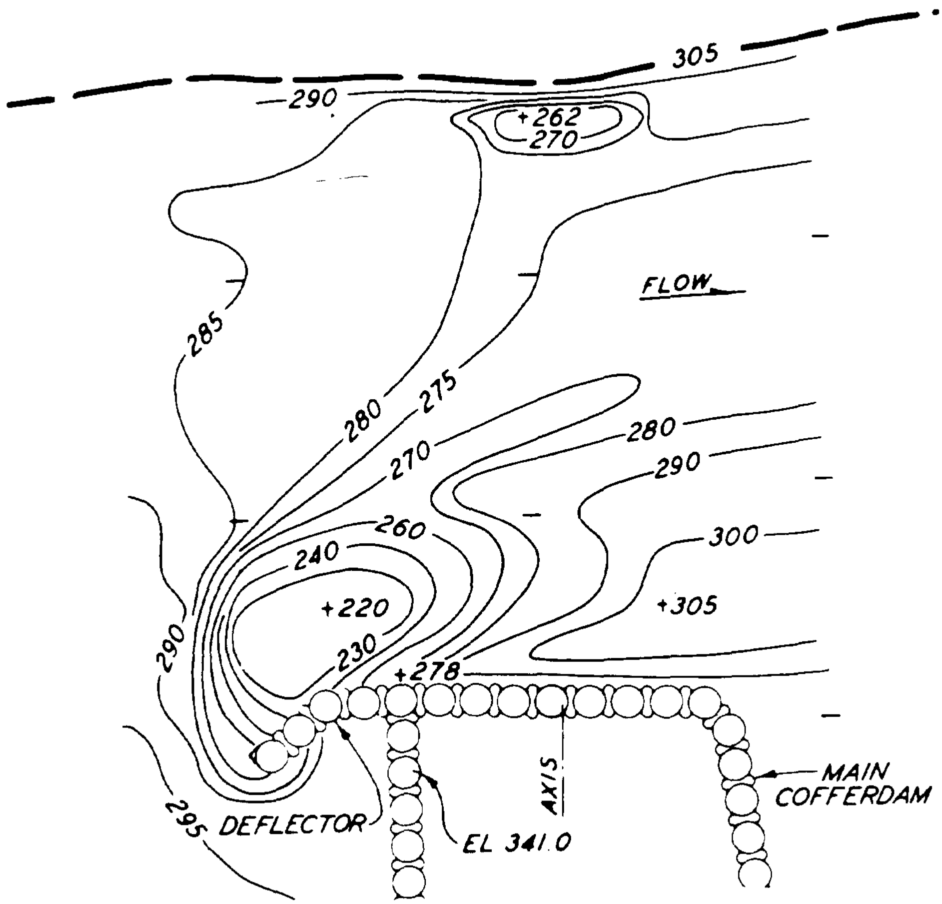


Figure 11-1. Typical scour pattern with deflector

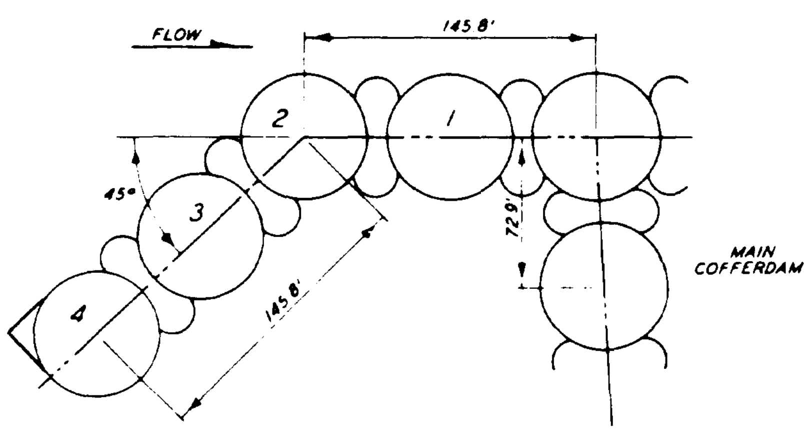


Figure 11-2. Cofferdam deflector

CHAPTER 12

OTHER FACTORS TO BE CONSIDERED

Section I. Effects of Locks and Dams on Sediment Movement

12-1. Spillway Operation. The movement of sediment in a channel with navigation locks and dams is affected by operation of the spillway gates and varies with river discharge and location within the pool. When discharges are such that normal upper pool is maintained or exceeded without gate control, open-river conditions prevail and the spillway gates are in a raised position. With this condition, the water-surface slope is nearly parallel to the bed with sediment movement occurring through the entire pool and through the dam over gate sills having crest elevation at or near the elevation of the streambed. As the discharge decreases to below the maximum required to maintain a normal upper pool elevation at the lock, the gates are closed in increments as required to maintain the minimum level of the pool. Closure of the dam gates produces a backwater effect and a reduction in the velocity of currents moving toward the dam. Because of the reduction in velocity, deposition begins at the dam and moves progressively upstream as the backwater effect continues to increase with decrease in river discharge and increase in the amount of gate closure. While deposition is occurring in the lower reach of the pool, sediment movement in the upper reach could continue until the backwater effect extends upstream to the next dam.

12-2. Hinged Pool Operation. The point of deposition will vary with river discharge and the amount of gate closure. If the discharge remains relatively constant for a considerable period of time, sufficient deposition could occur at a given location to require maintenance dredging in critical reaches. The location of deposition or scour can be controlled to some extent by variation in the normal pool level during critical flows. This operation, referred to as "hinged pool operation," would involve lowering the pool several feet (usually 2 to 5 feet) below normal upper pool during critical flows where adequate depths are available at the upper end of the pool and then raising the pool to extend the backwater effect above the critical reach. The amount of lowering should also consider the effect of the increase in velocities on navigation. This operation is also used in anticipation of powerhouse releases or rises in river stage and discharge upstream; this maintains water level at or below normal upper pool level longer and reduces the amount of stage variations.

12-3. Open-River Conditions. When the river discharge increases to above that required to maintain the minimum pool level, open-river conditions prevail and there is movement of sediment through the entire pool; the movement of sediment is generally greater in the lower reach because of the reduced cross-sectional area produced by the deposition. During open-river flows, deposition occurring over the gate sill and within the stilling basin should not affect gate operation since velocities over the sill are increased as the gates are closed. Since movement of sediment toward the dam varies inversely with the amount of gate closure, there would not be any serious tendency for sediment to deposit against fully or partially closed gates.

12-4. Depths in Upper Lock Approach. Although the lock and dam structures are used to maintain a minimum pool level some 20 feet or more above the natural low-water plane, depths in the upper lock approach cannot always be maintained without regulating structures (fig. 12-1).

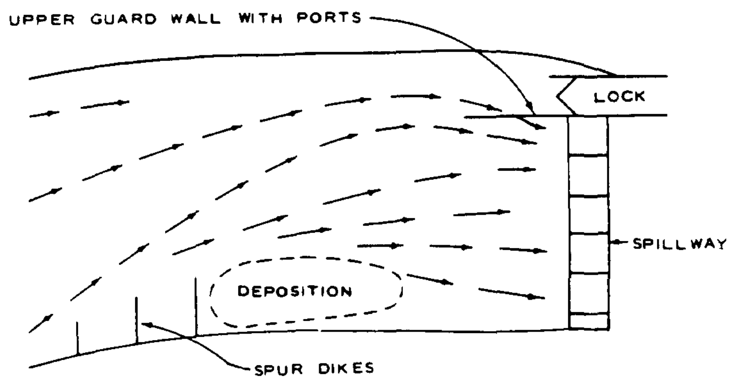


Figure 12-1. Training structures designed to maintain depths in the upper lock approach

This is particularly true if the alignment of the channel is such that there is a natural tendency for shoaling along the lock side of the channel. Manipulation of the gate opening would generally be ineffective in removing the shoal since most of the sediment movement occurs during open-river conditions. The tendency for shoaling on the lock side of the channel can be determined by studying the channel configurations before the structure is built and investigating the tendency of the channel to cross toward the opposite side of the river.

Section II. Harbors and Mooring Areas

12-5. Location. The development of commercial traffic on inland waterways will depend to a considerable extent on the availability of adequate mooring areas, fleeting areas, and docking and harbor facilities. In many cases, docking and harbor facilities are provided as part of the development or improvement of the waterway for navigation in cooperation with local interest. Unless these facilities are carefully planned, hazardous conditions could exist, particularly when placed along the bank adjacent to the main channel of the waterway. As a general rule, these facilities should not be placed close to lock approaches, on the concave side of a sharp bend, just upstream of a bridge, or where the channel tends to be of limited width.

12-6. Inland Harbors. When suitable areas for docking facilities along the streambank are not available, harbor areas are provided inland from the channel. These areas might be offsets in the bank line, lower reaches of tributary streams, old bendway channels, or an excavated area landward with a connecting entrance canal. The design of the harbor facilities should consider the traffic using the facilities, currents, ice and debris, movement of sediment, and effects of changes in river stages.

12-7. Harbor Entrances. When an opening is provided in the bank line, there is an abrupt change in the width of the channel and a tendency for shoaling in the expanded area. Shoaling in harbor areas or entrances to harbors can be a serious problem because of dredging cost, lack of suitable disposal areas, environmental factors that have to be considered, and interference with traffic. The tendency for shoaling will be greater when the entrance is placed on the convex side of a bend and increases with an increase in the size of the opening in the bank line.

12-8. Effects of Currents. Normally, tows entering the harbor area have to make a turn from the river channel. A downbound tow making the turn toward the entrance will tend to have its stern rotated downstream by the currents and could be in danger of hitting the banks of the entrance canal. When velocities are substantial, it might be necessary for downbound tows to turn around and approach the entrance from downstream. Flaring of the entrance to provide for both upbound and downbound tows would increase the tendency for crosscurrents in the entrance and the tendency for shoaling (fig. 12-2). In sediment-carrying streams, it is generally better to angle the entrance channel toward the downstream (fig. 12-3). With this alignment, the tendency for shoaling will be reduced and upbound tows can approach the entrance from along the adjacent bank in a direction nearly parallel to the alignment of the currents. When structures are required to prevent shoaling, it is generally not practical for downbound tows to enter the harbor without reversing their direction (fig. 12-4).

12-9. Old Bendways. Development of harbors in old bendway channels bypassed by a cutoff will generally require the closure of one end of

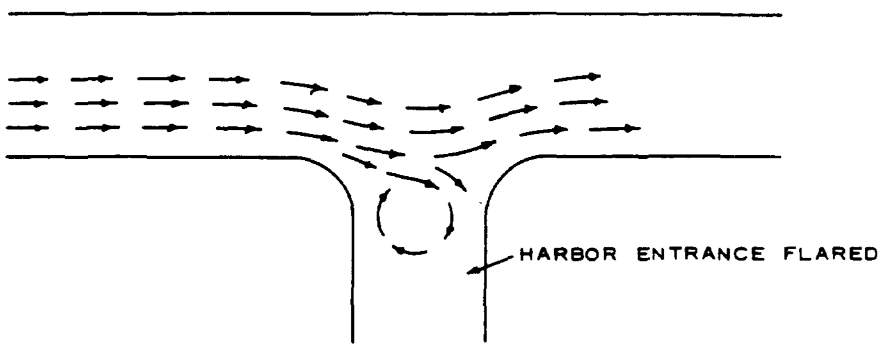


Figure 12-2. Currents with flared entrance

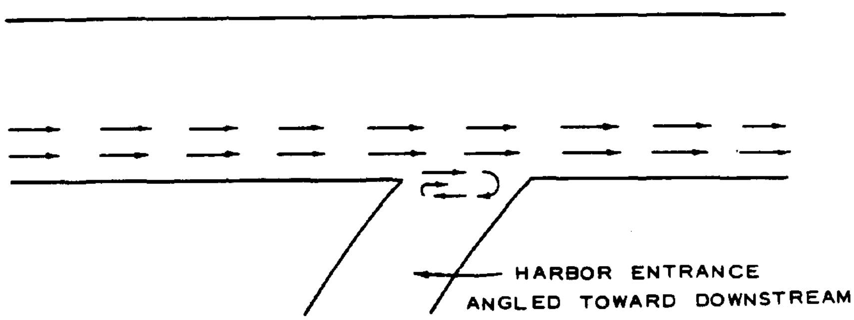


Figure 12-3. Currents with entrance angled downstream

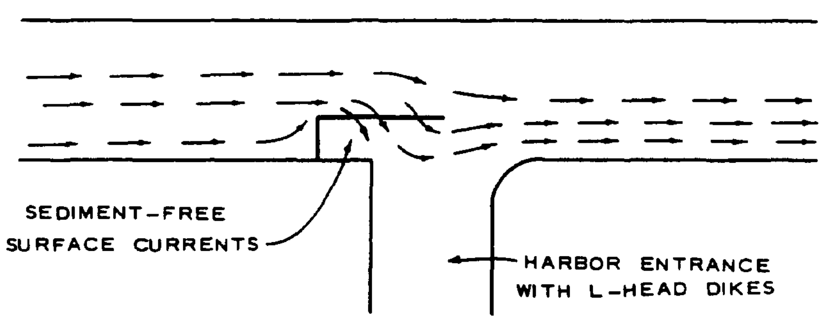


Figure 12-4. L-head dike used to reduce shoaling in harbor entrance

the channel (usually the upper end) and structures such as stepped-up dikes, L-head dikes, or wing dikes similar to those used in lower lock approach to reduce or eliminate the tendency for shoaling in the entrance at the lower end of the bendway (fig. 7-1).

12-10. Harbor Design Guidance. The principal features to be considered in the design of harbors are entrance and access channels, turning basin, mooring facilities, and loading and unloading facilities. Factors to be considered in entrance location and configuration are the effects of currents, wind, and shoaling problems, traffic congestion, visibility, direction from which most of the traffic is expected to approach the harbor, and maneuvering required. Access channels should provide the width needed for safe transit of the traffic anticipated based on one-way or two-way traffic in straight or curved channels, currents caused by variations in river stages, tides, local drainage, wind effects, and structures or equipment moored along the banks. Turning basins should be large enough to permit the size tows using the harbor to change directions without endangering equipment moored in the harbor or harbor facilities and without excessive maneuvering. The size and shape of the turning basin should be a matter of judgment based on the size of tows using the harbor, type of commodity handled, traffic congestion anticipated, safety, and efficiency.

Section III. Ice Problems

12-11. Effects on Navigation. Navigation on some of the northern waterways has been suspended annually and others have been affected periodically because of the heavy ice accumulations and their effect on traffic and the operation of facilities such as locks and dams and spillways. However, in recent years, the navigation season on some waterways has been extended and efforts will be continued to provide year-round navigation insofar as practical.

12-12. Effects on Structures. In addition to its effects on navigation, ice can cause damage to training and stabilization structures and mooring and docking facilities along the banks of the stream. Ice accumulation in lock approaches tends to block the entrance to the locks and could affect the operation of the lock gates. When a guard wall with ports is provided, ice and drift will tend to move into the lock approach and be trapped between the guard wall and adjacent bank and might have to be moved out or passed through the lock before traffic can be accommodated. Ice accumulation against partially closed spillway gates could render the gates inoperable and could result in flooding or overtopping of the dam and lock walls. The same effect could occur when ice jams or gorges develop in reaches between locks and dams or in open-river channels.

12-13. Design Considerations. The probability of ice formations and the movement of ice and debris should be considered in the design of spillways, locks and dams, channel alignment and dimensions, and necessary training and stabilization structures. Some of the provisions that might be considered are:

a. Air bubbler screen or ice boom designed to divert ice and debris away from the lock approach.

b. High air flow screens in gate recesses.

c. Lock emergency gates designed and maintained for passing ice and debris.

d. Means of preventing ice formation in the area of the lock miter gates, the lock filling and emptying valves, lock walls, and emergency bulkhead latching devices.

e. Protection from excessive scour downstream of the spillway during uneven operation of the gates to pass ice.

f. Heating cables or pipes in lock walls.

g. Chemical coating of lock walls to reduce ice adhesion.

h. Elimination of sharp bends in the channel and constricted reaches where ice jams or gorges might develop.

i. Provisions for raising and lowering of the upper pool above and below the normal pool level during low flows.

Section IV. Increasing Capacity of Existing Waterways

12-14. General. The capacity of existing waterways to handle modern traffic is often limited by the sizes of the available locks, lock operating facilities, navigation conditions in lock approaches, effects of adverse currents, limited channel dimensions and bridge clearances, location of docks and tow assembly areas within the approaches, and need for passage of small boats and pleasure crafts. Considerable increase in the capacity of some waterways can be accomplished by eliminating hazardous conditions, need for excessive maneuvering, and need for the temporary closure of the project because of accident or maintenance.

12-15. Modification of Locks. The sizes of locks and conditions in the lock approaches can be major factors affecting the capacity of a canalized waterway. When the locks are too small to accommodate the size tows using the waterway, multiple lockages and in some cases changes in the makeup of the tows will be required. The capacity of existing locks can often be increased by: modifications that would reduce lock filling and emptying time and time required to open lock gates; modification of lock auxiliary walls; providing accessible mooring facilities for waiting tows or sections of tows that cannot be accommodated in the lock; providing towing mechanisms or tenders to assist tows or section of tows through the lock; eliminating adverse conditions in the lock approaches; providing special facilities for pleasure crafts; and traffic regulation. Other alternates are enlarging the existing lock or construction of an additional lock large enough to accommodate existing traffic and traffic that can be reasonably anticipated.

12-16. Lock Approaches. The capacity of existing locks can be increased in many cases by modifications designed to eliminate hazardous conditions in the lock approaches and the effects of adverse currents which require considerable maneuvering of the tows before a satisfactory approach to the lock can be made. Safe navigation conditions in the approaches would permit the passage of larger and heavier loaded tows up to the full capacity of the lock and reduce downtime that might be caused by accidents. The modifications that can be made in the lock approaches and benefits obtained will depend on conditions at each lock and might include one or more of the following: realignment of the channel upstream and downstream; training structures designed to improve the alignment and velocity of currents; additional maneuver area; modification or extension of lock guide or guard walls; elimination of obstruction within the approach channel; mooring or protective cells; elimination of ice and debris from the lock approach; and reduction or elimination of any adverse effects from lock emptying and filling or powerhouse operations. Model studies can be invaluable in determining the conditions affecting navigation and in developing the most effective and economical solutions.

12-17. Lock Replacement or Addition. The capacity of some waterways cannot be increased substantially without the enlargement of the existing locks, construction of additional locks, or complete replacement and/or relocation of some of the lock and dam structures. The enlargement of existing locks would not be practical in most cases where traffic has to be maintained or where the existing structures have deteriorated to such an extent that cost of repairs or rehabilitation would be excessive. In such cases an additional lock or a complete replacement structure would be required.

12-18. Modification of Channel Dimensions. The capacity of existing channels is affected by the dimensions and alignment of the channel, velocity and alignment of currents, shoaling tendencies, and obstructions such as limited bridge clearances and accumulations of ice and debris. The draft of tows using a waterway will depend to a large extent on the controlling depths available. Increasing depths in critical reaches can increase tonnage considerably by accommodating tows with greater draft. The width of a channel will have some effect on the size of tows and whether one-way or two-way traffic can be accommodated. The capacity of a waterway can often be increased by increasing the width of the channel, particularly in bends and in reaches where sharp turns have to be made or maneuvering is required.

12-19. Current Alignment. The alignment of the channel and adverse currents can cause delays and contribute to accidents. Improvement of the alignment of the currents with respect to the alignment of the channel can eliminate the need for maneuvering and provide for adequate sight distance.

12-20. Bridges. Bridges and other structures with limited vertical and horizontal clearances can contribute to accidents and delays. Capacity and safety of the waterway can be improved in some cases by realigning the channel approaching the bridge, improving the alignment of currents upstream and downstream of the bridge, use of guide walls or fenders on the piers, or modification or replacement of the bridge.

Section V. Special Design Features

12-21. Special Features. Some special features that could have a significant effect on navigation conditions, operation and maintenance of the waterway, and/or cost of the project are discussed below.

12-22. Debris Control. Substantial amounts of floating debris can hinder lock operation and present a hazard to navigation. The usual debris disposal method is to pass it over the spillway which only presents a rehandling problem downstream. An alternative is to provide land disposal areas for debris at each project. Booms and workboats can direct debris to a shore pickup area. Air bubblers have been used successfully to keep debris out of lock miter gate recesses.

12-23. Standardization. Considerable economy can be achieved by standardization of some features of a project which would reduce design and procurement cost and require fewer replacement parts. An example is the Red River Waterway where spillway gate widths are the same for Locks and Dams 2, 3, 4, and 5. This allows interchangeability of spare tainter gates. Also, fewer maintenance bulkheads are needed to service the four projects.

12-24. Emergency Closure. All projects should have a contingency plan for access to spillway gates and lock gates so closure can be made in case of an accident. This closure is particularly important at high-lift locks and where there exists a high risk to downstream users. Closures can be made by stoplogs placed by cranes. If closure is desired under flow conditions, the crane must operate from a spillway bridge or lock wall. Also, bulkheads must be designed for placement in flowing water. If closure is to be made after the upper pool is lost, bulkhead placement can be made by a barge-mounted crane. This closure method requires that the upper lock gate sill and approach channel be lowered to an elevation where an upbound floating crane can reach the upstream dam face. Other closure methods for locks could be: inflatable dam, submerged tainter gate, or submerged vertical-lift gate.

12-25. Impact Barriers. During the period 1968 to 1977 there were in excess of 350 reported collisions of barges with miter gates. Both repair costs and lost navigation benefits were considerable. One method of reducing the chance of these collisions is to provide impact barriers. Barriers should be provided at locks when a gate failure would cause loss of life or the repair cost and lost navigation benefits would justify the barrier cost. If barriers are considered necessary, they should be designed to withstand the impact of a full-size loaded tow traveling at a reasonable speed. Some of the provisions for the prevention of accidental damage to miter gates that should be considered are as follows:

a. Double Lock Gates. Double gates have been the traditional safeguard but wire rope or nylon net barriers should be considered. High-lift locks could have a lower guard miter gate with the bottom portion removed. This would allow returning the guard gate to the recessed position when the tow dropped below the gate. This type of gate would not require any expensive lock lengthening.

b. Concrete Beam. Another concept is to build a concrete beam across the lock, downstream from the lower miter gate. The gate would seat against this beam in the mitered position. When the chamber is empty, the gates would open and the tow would pass under the beam when exiting the chamber. If a barge collided with the lower gate, the impact load would be transferred to the beam with little damage to the gate. The beam could also serve as a structural member and a bridge for equipment movements.

c. Lift and Submergible Gates. A properly designed vertical-lift gate and submergible tainter gates can withstand a much greater collision than a miter gate and may be attractive when compared with a miter gate plus impact barrier combination.

12-26. Water Conservation. The competing interest for water can often provide a compelling case for conserving lockage water. Some water conservation measures for a navigation project with locks and dams could be:

a. Water Saving Basins. Provide a basin adjacent to the lock that can be filled with emptying water from the upper elevation of the chamber. When the basin is full, the lock discharge water is directed to a lower basin or into the lower approach. During filling, the water is drained (by gravity) into the lock thus saving water (volume equal to the basin) from being withdrawn from the upper pool.

b. Intermediate Gates. The chamber can be divided into half or thirds by providing intermediate closure gates between the upper and lower gates. This conserves water by filling only a part of the chamber when short tows or pleasure craft lock through. Modifications to the filling and emptying system are needed to assure safe and efficient operation for both partial chamber or full chamber lockages.

c. Pumpback. Lock water can be returned to the upper pool for reuse by pumps.

12-27. Mooring Facilities. Mooring facilities should be provided upstream or downstream from a lock, if waiting barges would present a hazard to navigation or the project. These structures could be sheetpile cells or ported walls with rings or mooring bits or grappling hooks anchored on the bank. Locks with adverse currents or long waits for lockage should consider mooring facilities.

Section VI. Effects of Surface Waves

12-28. Waves Generated by Traffic. Surface waves of substantial size can be generated by tows and by recreational boats. Waves generated by traffic can adversely affect equipment and barges moored along the banks and the stability of the material forming the banks. The size of the waves reaching the bank or moored equipment will vary with the distance from the bank or equipment that the tow passes; the type, size, and draft of the tow; speed of the tow; and depth of the channel. Waves will tend to increase hawser stresses of barges moored along the bank and could cause them to break loose and endanger traffic or structures downstream. The size of waves and their effects can be reduced by limiting the size and speed of the tows using the waterway and by preventing tows from moving within a fixed distance from the banks or moored equipment. Fast-moving small recreational boats passing close to the banks or mooring areas can produce waves that are more objectionable than those created by most tows.

12-29. Wind Waves. Waves generated by wind would depend on wind velocity and the length of fetch in the direction of the wind. Except during storms or atmospheric disturbances, wind waves do not constitute a hazard to navigation in rivers and canals but can have a serious effect in large lakes and bays with long fetches and high prevailing winds. Winds can also affect the maneuverability of tows, particularly tows with empty barges. Wind blowing in an upstream direction will tend to produce a higher wave than when blowing toward the downstream. Wind waves on rivers and canals are usually small but can be continuous for long periods and can have some effect on bank erosion.

12-30. Prototype Measurements. The size of traffic-created waves on the Ohio River and some of their effects on the environment were measured and evaluated by the U. S. Army Engineer District, Huntington, with the assistance of the U. S. Army Engineer Waterways Experiment Station (WES) and special consultants. These measurements covered a large number of vessels of different sizes moving upstream and downstream at different speeds and distances from the bank line. Although a final report on the results had not been completed, preliminary analysis and evaluation of the results indicated the following general conclusions:

a. The size of waves approaching the bank increased with the size, draft, and speed of the tow and decreased with increase in the distance from the bank.

b. Waves created by small recreational vessels can be as large or larger than those created by towboats.

c. Wave sizes tended to be greater with greater depth (during high water) and tend to become smaller approaching a sloping bank or beach than when approaching a vertical bank.

d. The effects of traffic on the physical and biological components of the Ohio River were generally insignificant in comparison with ambient and natural changes. Changes produced by traffic were generally small and of short duration. The largest wave measured had a height of

3.3 feet produced by a towboat with nine loaded barges moving downstream at a speed of about 12 miles per hour and 200 feet from the water's edge. The measurement was made about 15 feet from the water's edge where the depth was about 5.5 feet. It should be noted that the Ohio River is wider and deeper than most streams or inland waterways. Conditions in restricted waterways could be considerably different.

CHARTER 13

ENVIRONMENT

Section I. Existing Regulations

13-1. Background. With the passage and implementation of the National Environmental Policy Act (NEPA) of 1969 (Public Law (PL) 91-190), environmental impact assessments of water resource projects under the U. S. Army Corps of Engineers and other Federal agencies assumed a greater level of importance. Previously environmental assessments were controlled by internal regulations and were usually not distributed or reviewed outside the agency; subsequently, NEPA established a broad national policy directing Federal agencies to maintain and preserve environmental quality.

13-2. Environmental Impact Statement. Section 102(a)c of NEPA requires all Federal agencies and officials to (a) direct their policies, plans, and programs to protect and enhance environmental quality; (b) view their actions in a manner that will encourage productive and enjoyable harmony between man and his environment; (c) promote efforts that will minimize or eliminate adverse effects to the environment and stimulate the health and well-being of man; (d) promote the understanding of ecological systems and natural resources important to the nation; (e) use a systematic and interdisciplinary approach that integrates the ecological, social, cultural, and economic factors in planning and decision-making; (f) study, develop, and describe alternative actions that will avoid or minimize adverse impacts; and (g) evaluate the short- and long-term impacts of proposed actions.

Section II. Recent Research and Research in Progress

13-3. Dredged Material Research Program (DMRP). The DMRP was completed by the WES in 1978. The objective of the program was to determine the environmental effects of dredged material disposal and to develop methods for eliminating or minimizing any adverse effects. Results of the DMRP were synthesized into a series of 23 reports (and corresponding EM's) that should be referred to when evaluating and/or minimizing impacts associated with dredged material disposal. (See Appendix A. References.)

13-4. Dredging Operations Technical Support (DOTS). The DOTS Program was established in 1978 at the conclusion of the DMRP to assist all Corps elements in the implementation of DMRP results. The program has the responsibility to maintain the capability at WES of responding to requests for assistance from the Corps elements on all environmental problems associated with dredging, dredged material disposal, and habitat creation. Continued monitoring of DMRP-developed engineering and operations methodologies are also under DOTS management. Regulatory functions research under DOTS include research in support of criteria development and wetlands identification/delineation.

13-5. Environmental and Water Quality Operational Studies (EWQOS). The principal objective of EWQOS, initiated in 1977, is to provide new or improved technology for planning, design, construction, and operation of CE Civil Works projects to meet environmental quality objectives in a manner compatible with authorized project purposes. Because there presently is a significant lack of data regarding the environmental effects of CE activities on navigable river systems, one of the major areas of investigation under EWQOS is providing information on the environmental effects of various waterway activities plus developing resource management plans and new or improved design and operation guidance to maximize environmental benefits or attain environmental quality objectives.

Section III. Factors to be Considered

13-6. Background Environmental Considerations. Problems to be considered in the development or improvement of waterways for shallow-draft navigation include the potential adverse effects of the project on environmental quality. Some of the factors that could affect the environmental quality of a waterway are:

a. Excessive Sedimentation. Bank erosion potential, adjacent land use practices, and general soil characteristics should be given consideration during site selection to prevent undesirable environmental effects from sedimentation and to minimize or eliminate the need for maintenance dredging. The need for reduction of bank slopes or other means of protection such as use of vegetation, gabions, or rock riprap to reduce the tendency for erosion from currents and waves should be considered. Old bendways cutoff during construction are becoming more important as aquatic habitats. Such areas function effectively as sediment traps and may require special treatment to maintain their effectiveness as desirable aquatic habitats. Disposal areas located adjacent to the main stream or tributaries should be designed and operated such that the effluent meets appropriate Federal and State water quality standards for suspended sediments.

b. Resuspension of Contaminants. Construction and maintenance dredging could cause the resuspension of contaminants. This is most likely to occur in waterways that have been used in the past as carriers of industrial, agricultural, or municipal wastes. Existing and past industrial and agricultural practices within the watershed should be examined and, if deemed necessary, appropriate sediment and water chemistry conducted to evaluate the potential impacts of any resuspended contaminants upon the aquatic environment.

c. Increased Water Temperature. Care should be taken to prevent the unnecessary removal of woody vegetation adjacent to the waterway. If such removal is a necessity, it may be possible to remove such vegetation from only one side of the waterway so as to maximize the shading effect.

d. Water Table Effects. Canalization and subsequent pooling of water behind a lock and dam may result in changes in the water table, thus changing the vegetation and the habitats available.

e. Excavated Material. A major concern in many project areas will be the methods used to remove and treat excavated and dredged materials, depending on the nature of the materials and their potential for releasing contaminants as discussed in paragraphs 13-6a and b above.

f. Impacts on Aquatic, Wetland, and Terrestrial Habitats. The route selected, construction activity, and management and operation of the project are all likely to have some adverse effects on biological habitats. The project and alternatives available should be evaluated to determine if any of the adverse effects could be eliminated or at least minimized. It might be possible to provide alternate habitats for certain species that are seriously affected.

g. Interruption of Migratory Routes. Evaluation of the use of the streams and adjacent terrestrial habitats as migration routes for aquatic and terrestrial animals is an important consideration during the planning process. Critical routes should be avoided when practical or provisions should be made for allowing alternate passage of the affected animals. Construction and/or maintenance activities could also be scheduled in such a manner as to avoid peak migration periods to reduce impact.

h. Modifications of Riparian Habitats. Bottomland hardwood forests are regarded as an important, though rapidly disappearing riparian habitat. Alternatives to the removal of existing natural riparian habitat should be developed so as to lessen such an adverse impact. Plans for revegetation should be developed where habitat modifications are necessary.

i. Disruptions of Breeding or Nursery Areas. Certain areas such as Cypress or Tupelo swamps, marshes, and Oxbow Lakes along rivers and streams are more critical than others for breeding, nursery, or nesting areas for aquatic, terrestrial, or arboreal animals. Particular care should be taken to identify such areas and arrive at suitable alternatives to the disruption of such habitat.

j. Increased Turbidity. Turbidity is an indication of suspended and colloidal materials in the water. Continuing high turbidity levels in a waterway over preproject conditions could adversely affect aquatic species. Measures such as construction of sediment traps, reseeding of construction areas, and construction of channel bypasses to prevent project contributions to increases in turbidity should be carefully considered in all phases of project design.

k. Impacts upon Wetlands. Our nations wetlands have been diminishing rapidly during the past half century. Such wetlands, in addition to serving as valuable habitat for diverse fish and wildlife communities, often are valuable for natural purification of polluted or contaminated waters. Wetlands also serve to eliminate severe changes in the water table, and often are highly regarded aesthetically. It may be possible, with proper consideration, to enhance wetland habitats along waterways and prevent unnecessary losses to existing wetland areas by using dredged material to create additional wetland areas.

l. Changes Associated with the Formation of Bendway Cutoffs. Many shallow-draft waterways projects result in the formation of bendway cutoffs by channelization for realignment of the navigation channel. Such areas in the past often served as a repository for excess dredged materials. This is no longer an acceptable practice, and furthermore, the potential value of such bendway cutoffs as aquatic habitat and recreation areas is being frequently included in the planning and design. Ongoing research is seeking to develop definitive guidance for the use and management of these cutoffs. These areas are often subject to rapid sedimentation and filling by bedload materials, and some structural measures are often required to prevent the premature loss of these areas as aquatic habitats.

CHAPTER 14

COST ANALYSIS

14-1. Cost Optimization. Engineering is a science that has as its purpose satisfying the wants and needs of people. In accomplishing this objective, the aim of the engineer should be to attain maximum results in the most economical manner. This cost optimization should provide the basis for selecting a project level of protection or evaluating alternative designs once project functional adequacy and safety are assured. In other words, only after design criteria have been achieved (minimum level of protection) can cost optimization be applied.

14-2. Elements. The elements that are to be considered in an economic optimization or life cycle analysis are:

a. Project economic life.

b. Construction cost for various levels of protection.

c. Maintenance costs for various levels of protection.

d. Replacement costs for various levels of protection.

e. Benefits for various levels of protection using probability analysis.

14-3. Effects of Protection Level. The construction cost will generally increase as the level of protection increases. Maintenance generally decreases as the level of protection increases. Replacement is less frequent and present worth annual costs are less as protection level increases. Benefits generally increase as protection level increases because frequency of losses (both time and property) decreases.

14-4. Economic Life. A Corps of Engineers project economic life is generally 50 years; however, some projects such as cofferdams or temporary sheet-pile locks can have shorter project lives. Once the economic project life is selected the level of protection to design for is needed. This level of protection or condition to design for is related to the occurrence of physical events such as river discharge, wind speed, or ice thickness. The severity or magnitude of these events has a statistical distribution that can be ordered into a frequency of occurrence. The frequency is converted to exceedance probability and plotted against the level of protection as shown in figure 14-1.

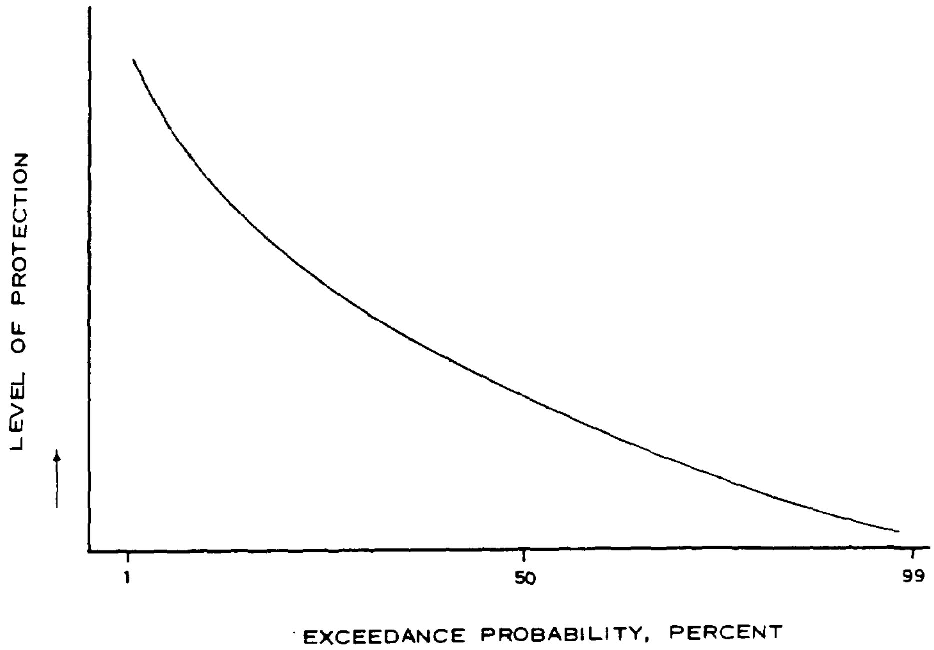


Figure 14-1. Probability versus protection level

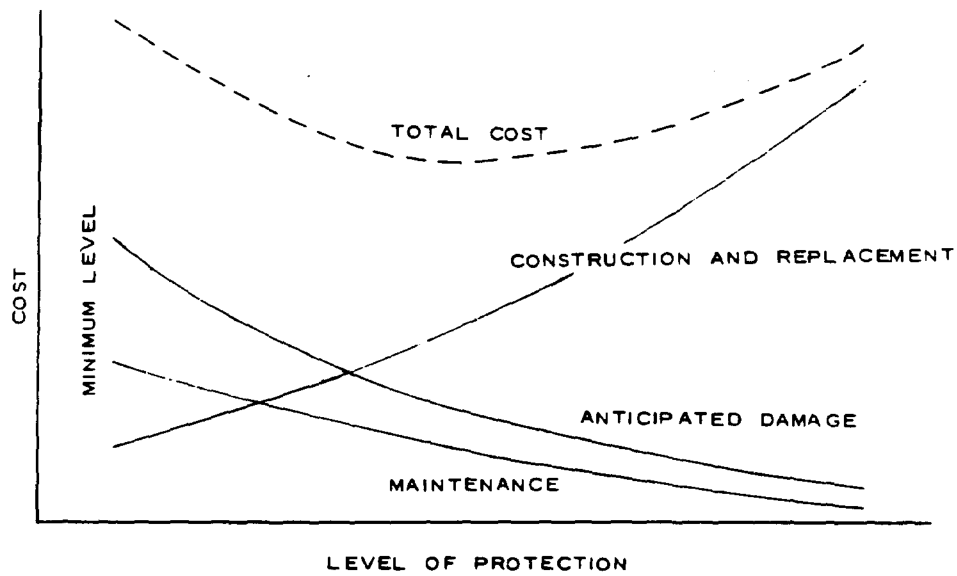


Figure 14-2. Project cost curves

14-5. Annual Damage. The expected annual damages are computed using standard methods. The anticipated annual damages can be computed by multiplying the expected annual damages by the annual exceedance probability for various levels of protection. This anticipated annual damage value is added to the amortized construction cost, annual maintenance cost, and present worth amortized replacement cost to obtain the total project cost. A series of these total project cost estimates for various levels of protection will provide a total cost curve as shown in figure 14-2. The optimum design is indicated by the lowest point on this curve.

14-6. Total Cost. The total cost curve may be fairly flat at the minimum point. If this occurs, it may be prudent to select a higher design level. A simplified life cycle cost analysis is presented in the following example problem.

Example Problem

Problem: Compare concrete side port filling lock with sheet-pile side flume filling lock.

Given:

a. 50-year project life.

b. 50-year life for concrete lock.

c. 25-year life for sheet-pile lock.

d. Lost benefits during replacement due to construction of adjacent lock $2,000,000/year for 4 years.

e. Sheet-pile filling time 20 minutes. Concrete lock filling time 8 minutes.

f. Average annual loss for slower filling is $1,500,000.

g. Interest rate 6 percent.

Find: Least annual cost lock using life cycle analysis.

Analysis: Step 1. Estimate initial construction cost.

Step 2. Compute present worth of replacement (using initial construction as equivalent dollar value for replacement).

Step 3. Estimate lost benefits incurred during construction of replacement lock.

Step 4. Compute present worth of lost benefits.

Step 5. Total present worth cost and amortize for project life.

Step 6. Estimate annual maintenance cost including lost navigation benefits during downtime.

Step 7. Estimate lost benefits for slower filling time lock.

Answer: The concrete lock has the least annual cost.

The project cost computations are presented below.

<u>Concrete Lock</u>

Initial Cost	$60,000,000
Annual cost of construction (crf - 6 percent - 50 years)	3,806,400
Annual maintenance including lost benefits for downtime (50-year average)	50,000
Total Annual Cost	$ 3,856,400

Sheet-Pile Lock

Initial Cost	$25,000,000
Replacement after 25 years (in today's dollar value) = $25,000,000	
Present worth (pwf' - 6 percent - 25 years)	5,825,000
Present worth of loss during replacement construction ($8,000,000 - pwf' - 6 percent - 25 years)	1,864,000
Total Present Worth	$32,689,000
Annual Cost (cfs - 6 percent - 50 years)	$ 2,073,800
Annual Maintenance including lost benefits for downtime	500,000
Annual lost benefit for slower filling time	1,500,000
Total Annual Cost	$ 4,073,800

crf = Uniform annual series, capital recovery factor

pwf' = Single payment, present worth factor

Chapter 15
Model Studies

15-l. General

The development of satisfactory navigation conditions in the approaches to locks and dams and in critical reaches requires a knowledge of existing conditions with all navigable flows, changes produced by structures or modifications, and effects of changes on conditions affecting navigation. Adequate data are seldom, if ever, available to permit a reasonable analysis of the conditions existing in a particular reach and time will usually not permit a detailed survey of the reach (some of the flows that should be considered might not be experienced for several years). Because of the complex nature of flow in natural streams, analytical studies to determine probable conditions from a particular plan of improvement are generally extremely difficult and inconclusive, even if sufficient field data on existing conditions were available. An example of how model studies modified an original design is shown in Figures 15-1 and 15-2. The modifications were needed to assure safe approach conditions.

15-2. Use of Model Studies

Channel and overbank configurations and flow conditions are never identical in any two reaches of the same or different streams; designs that prove satisfactory at one site might not be adequate at another. For this reason model studies have been used extensively in the development of plans for locks and dams, bridge modifications, channel realignment, construction sequence, and for the reduction or elimination of channel maintenance. As a result of model studies, designs have been simplified in many cases, with considerable reduction in the cost of the project, and in others, the cost had to be increased because of the indicated need for better conditions and facilities.

Section I
Physical Model Studies *

15-3. Optimum Design

Small, financially insignificant changes in design can sometimes make the difference between good and bad navigation conditions. Correcting undesirable conditions before the structure is built can result in the elimination of costly maintenance and remedial measures. By using model studies, alternate plans and modifications can be tested within a relatively short time with all flow conditions that can be expected. Also, the design and operating engineers can observe conditions resulting with a particular arrangement and satisfy themselves as to the adequacy of the plan. In many cases, navigation interests are invited by the sponsors of the study to observe demonstrations of the plans developed, to operate the model towboats and tow, and to submit comments and recommendations. Utilization of this procedure results in the final design being based on the results of a complete investigation and the opinions and evaluations of the best qualified design engineers, engineers familiar with model investigations of these types of problems, and engineers responsible for operation of the facilities and the towing industry.

15-4. Cost of Model Studies

The cost of model studies varies with area under study, characteristics of the streams, nature of the problem, and number of plans and alternate plans to be tested before an acceptable solution is developed. The cost of model studies has usually been less than 0.10 percent of the cost of the project, a small price to pay for the assurance that the most practical and economical design has been developed. Both fixed-bed navigation models and movable-bed sedimentation models are recommended for lock and dam studies on alluvial streams. Only fixed-bed models are generally required for streams carrying little or no sediment.

* ***Section II***
Numerical Model Studies

15-S. Numerical Models

a. Numerical modeling is a rapidly developing discipline that can be attributed to the general availability of fast, large-memory computers. A numerical model basically consists of a numerical algorithm developed from the differential equations governing the physical phenomena All numerical models require the study area to be discretized by a grid or mesh. Furthermore, testing the numerical results against a prototype data set (verification) is highly recommended.

b. Numerical models may be used to replace or supplement physical models. The following types of investigations can be studied with numerical models:

(1) Provide general circulation patterns for deep- or shallow-draft ship simulator studies.

(2) Determine shoaling and erosion characteristics.

(3) Address dredged material disposal issues and other water quality measures.

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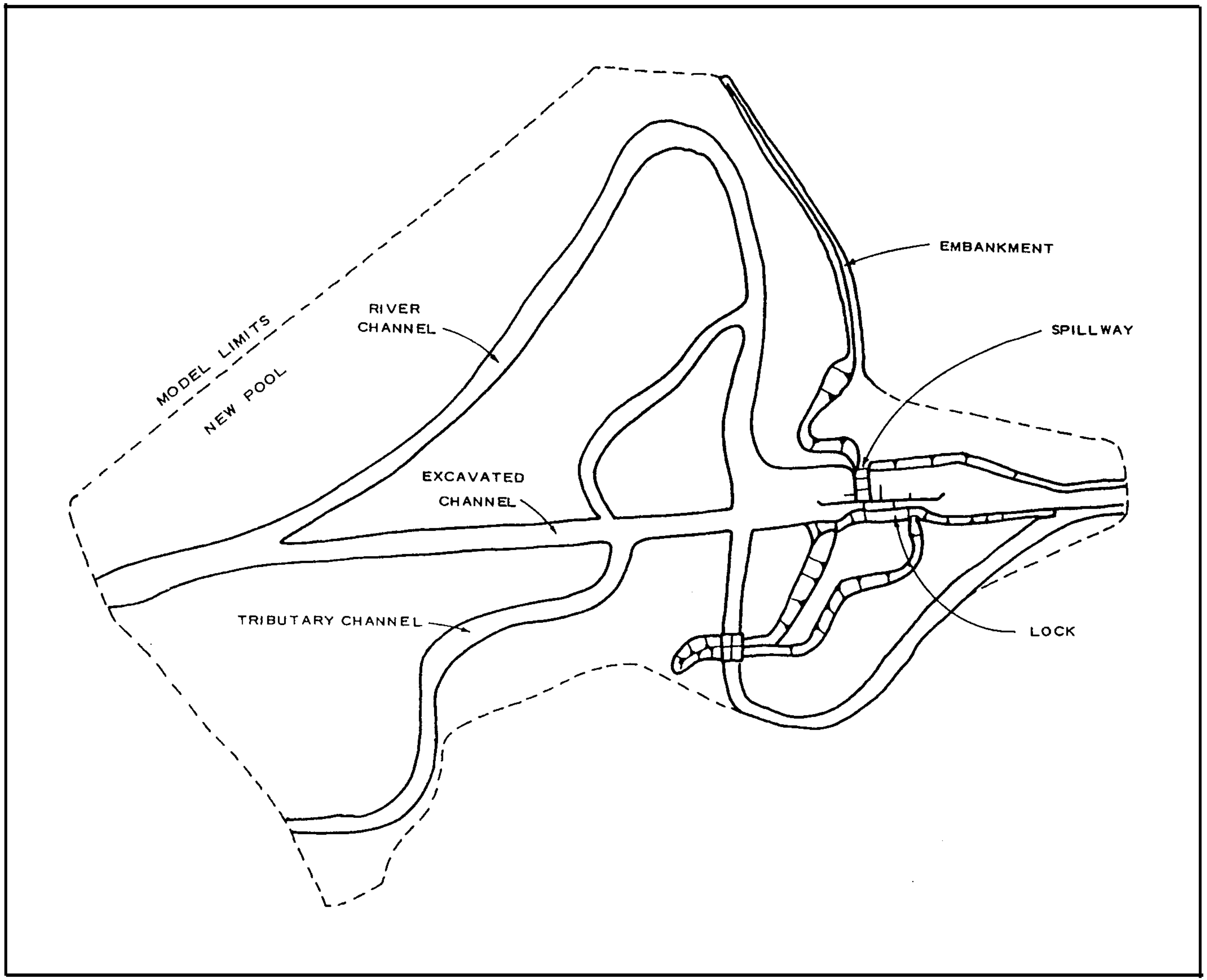

Figure 15-1. Original plan for lock and dam

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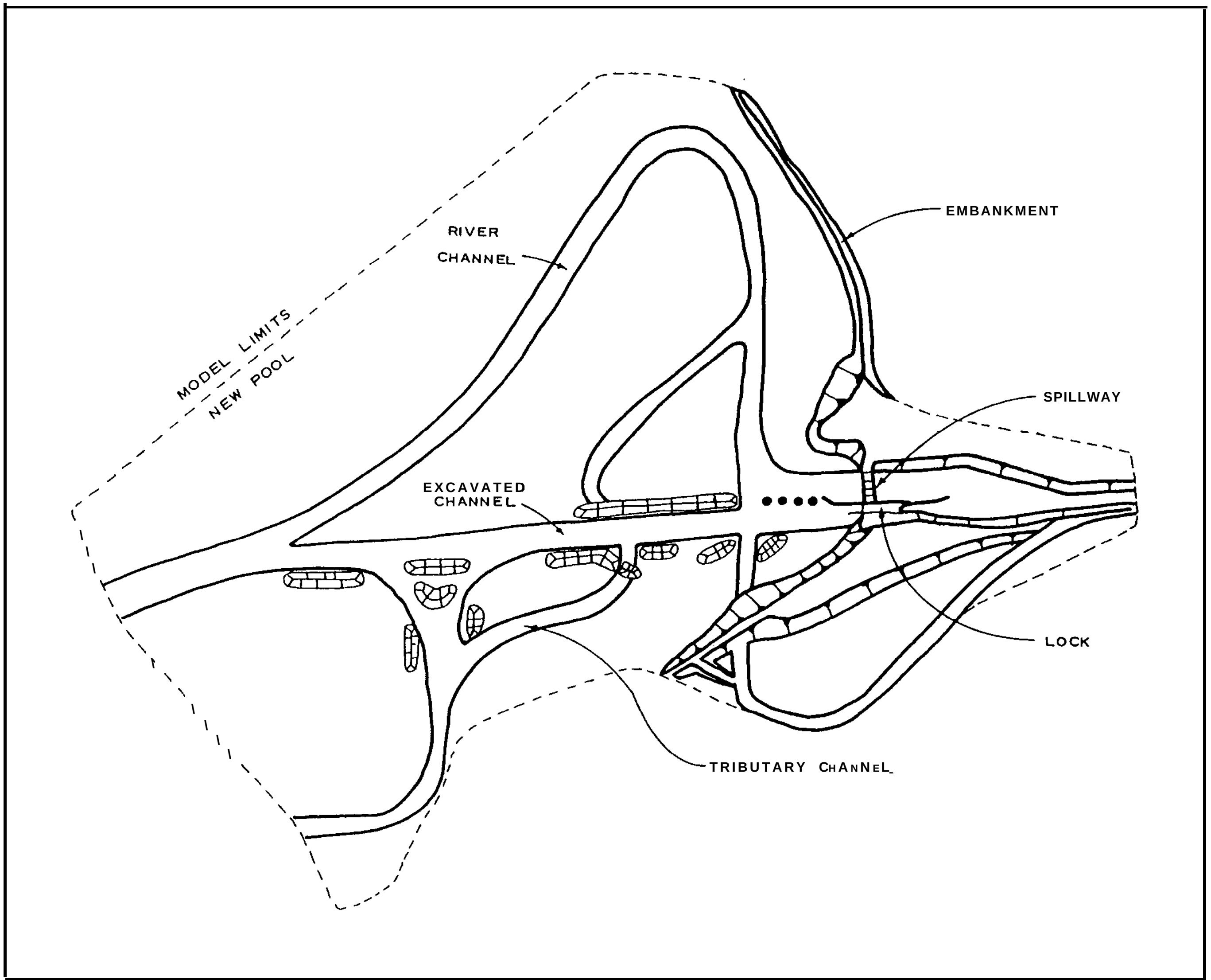

Figure 15-2. Improved plan based on model tests

* (4) Investigate salinity intrusion.

(5) Study wave penetration and harbor response.

(6) Evaluate training structure designs.

c. Numerous numerical models are available within the scientific community. These models differ in several ways: formulation, governing equations, and user friendliness, to name a few. Some numerical models have the ability to solve hydrodynamics and transport equations simultaneously while others are uncoupled.

d. The two basic numerical model formulations are finite difference and finite element. Finite difference is the easiest to conceptualize. A finite difference model approximates the calculus differential operators by differences over finite distances. This gives an approximation of the governing equations at discrete points. The finite element model approximates the mathematical form of the solution and inserts it into the exact form of the governing equations. After boundary conditions are imposed, a set of solvable simultaneous equations are created. The finite element solution is continuous over the area of interest.

e. The governing equations describe the physical processes that are being solved in the model. The dimensionality of the problem is dictated within these equations. These equations describe the physics of the problem For a hydrodynamic model these would include items such as friction, density, gravity, rotation of the earth, wind, rain, inflows, and outflows.

f. The term user friendly is an all-encompassing issue dealing with ease and efficiency of use. It addresses the process of creating a mesh, specifying the parameters within the computational domain, analyzing the solutions, generating presentation and report quality graphics, on-line documentation, and consultation support.

g. Several models are available within the U.S. Army Corps of Engineers **(USACE)** that have met the test of time. One such model is the TARS-MD numerical modeling system The multidimensional aspects of TABS-MD have expanded the capabilities of the system such that it has had hundreds of applications within the USACE. TABS-MD has been utilized by a multitude of private consulting firms and universities as well. It has a good reputation and a state-of-the-art graphical user interface that makes it one of the most user-friendly and efficient ways to conduct a numerical model study. Numerous technical reports and papers have been published on TABS-MD applications, the most recent of which are listed in Appendix A.

15-6. TABS-MD Numerical Modeling System

a. The TABS-MD is a collection of several generalized finite element models and pre- and post-processing utility programs integrated into a multidimensional numerical modeling system TARS-MD is suitable for use in solving hydraulics behavior, sedimentation, and transport problems of rivers, reservoirs, wetlands, estuaries and bays. Examples of past use include predicting flow patterns and erosion in a river reach constricted by a cofferdam, evaluating sedimentation rates in a deepened navigation channel (both riverine and estuarine), determining the impact of flood control structures on salinity intrusion, developing recommendations for a safe and cost-effective navigation channel design, and defining flow and sedimentation impacts to wetlands.

b. The system is designed for use by engineers and scientists who are knowledgeable of the physical processes that control behavior of waterways, but who may not be computer experts. TABS-MD offers a complete range of model study functions, including map digitization, mesh generation, modeling, and graphical display of numerical model results.

c. TABS-MD is currently operational on a wide variety of computer platforms, ranging from the CRAY super computer to the personal computer (PC). The numerical models and most of the utility programs are written in FORTRAN-77 and will soon be updated to FORTRAN-90. Plans am underway to modify the models to take advantage of parallel processor environments.

d. The system is maintained by the U.S. Army Engineer Waterways Experiment Station (WES), and includes two hydrodynamic models: RMA2-WES and RMA10-WES. In this context, the term hydrodynamic modeling is a general term intended to denote a body of water with a free surface such as a river. The first fundamental decision, prior to conducting a numerical model study, is to classify the study area in order to choose the appropriate numerical model. RMA2-WES is an appropriate choice for a far-field problem whose study area may be modeled with a two-dimensional (2-D) depth-averaged approximation. Otherwise, the modeling effort must employ RMA10-WES to incorporate the three-dimensional (3-D) aspects. TABS-MD permits an efficient numerical approach by incorporating multiple dimension concepts within a given mesh domain. For instance, a RMA2-WES application may use economical one-dimensional (1-D) calculations in some areas and 2-D ones within the primary area of interest. A RMA10-WES application may *

* use any combination of l-, 2-, and 3-D calculations with or without the transport options. Needless to say, the modeling effort can reach a high degree of complexity and computational burden with 3-D computations.

Two sediment transport options are available with the TABS-MD system SED2D is a 2-D finite element model that solves the convectiondiffusion equation with bed source-sink terms. These terms are structured for either sand or cohesive sediments. Cohesive deposited material forms layers, and bookkeeping allows layers of separate material types, deposit thickness, and age. SED2D uses the hydrodynamic solution generated by the RMA2-WES model. RMA2-WES and SED2D are uncoupled, therefore, a new geometry must be cycled back to RMA2-WES when the bed deposition and erosion patterns begin to significantly affect hydrodynamics. Work is ongoing to upgrade SED2D to accommodate all features of RMA2-WES, such as 1-D and marsh/wetland calculations. The other sediment transport option is to couple the sediment transport with the hydrodynamic calculation by using RMA10-WES. RMA10-WES includes a single-class fine-sediment transport with an associated layered bed with distinct densities and erodibilities for each layer. Changes in bed elevation are made during computations and are accounted for in the continuity equation.

f. There are two water quality transport options within TABS-MD as well. RMA4-WES is a 1-D and 2-D finite element model with a form of the convective diffusion equation with general source-sink terms. The model may transport and route up to six constituent substances, with or without decay. The model accommodates a mixing zone outside the model boundaries for estimation of re-entraimnent. RMA4-WES uses the hydrodynamic solution generated by the RMA2-WES model. RMA10-WES has the option to couple temperature, salinity, and/or sediment transport with the hydrodynamic calculations.

A recent research effort was conducted at WES to provide guidelines and help field offices conduct hydrodynamic numerical models to address both deep-draft and shallow-draft issues. The work emphasized RMA2-WES hydrodynamic applications since all navigation studies involve that aspect and most of the field offices have access to personal computers or workstations capable of running 2-D simulations. Furthermore, the WES ship simulator typically uses the RMA2-WES solution as input to define the currents for the simulator (Figure 15-3).

15-7. Example Navigation Applications using RMA2-WES Solutions

a. ***Charleston, SC. Estuary.*** The study was undertaken to evaluate and optimize proposed improvements including deepening the navigation channel from 40 to 45 feet, realigning and/or widening several fairways along a 5-mile stretch of the estuary, and locating a proposed seven-berth container terminal. The RMA2-WES simulation was conducted to provide currents to the WES ship simulator for several time-steps on both the ebb and flood portions of a spring tidal cycle. Figure 15-3 shows the WES ship simulator response track plot corresponding with one set of velocity vectors computed by RMA2-WES for the Drum Island reach of the study area. The study was an iterative process between the hydrodynamic model, the ship simulator model, and the SED2D sediment transport model, as indicated by the flowchart in Figure 15-4.

b. *Redeye Crossing near Baton Rouge, LA, along the Lower Mississippi River.* The study was undertaken to evaluate the effect of river training structures on vessels (both ships and tows) transiting the Redeye Crossing Reach. Studies included a TABS-MD RMA2-WES hydrodynamic model, the ship/tow simulator model, and a SED2D sediment transport model. Figure 15-5a and b show the WES tow simulator response track plot corresponding to one set of velocity vectors computed by RMA2-WES using the secondary flow corrector. Figure 15-5c shows the computational mesh used by the TABS-MD models. The study was an iterative process between the RMA2-WES hydrodynamic model, the ship simulator model, and the SED2D sediment transport model, as indicated by the flowchart in Figure 15-4.

15-8. RMA2-WES Hydrodynamic Model

RMA2-WES is a finite element solution of the Reynolds form of the Navier-Stokes equations for turbulent flows. Friction is calculated with Manning's equation, and eddy viscosity coefficients are used to define the turbulent exchanges. A velocity form of the basic equation is used with side boundaries treated as either slip or static. The model has a marsh porosity option as well as the ability to automatically perform wetting and drying. Boundary conditions may be water-surface elevations, velocities, discharges, or tidal radiation. Both steady and unsteady free-surface calculations for subcritical flow problems can be analyzed.

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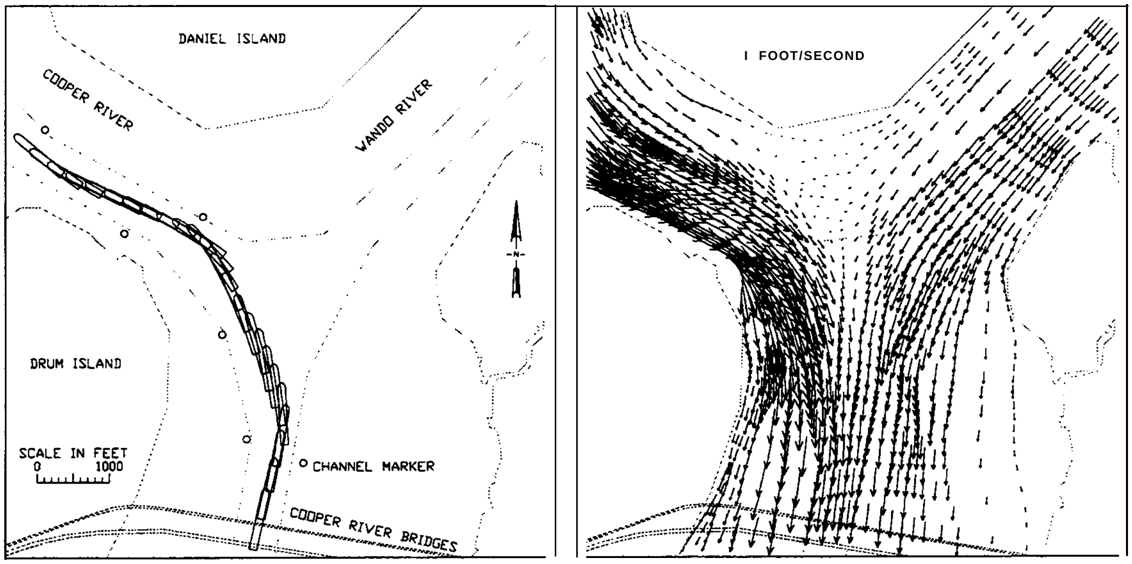

a. WES ship simulator track plot **b. RMA2-WES hydrodynamic solution**

Figure 15-3. The Cooper River, Charleston, SC, channel realignment study

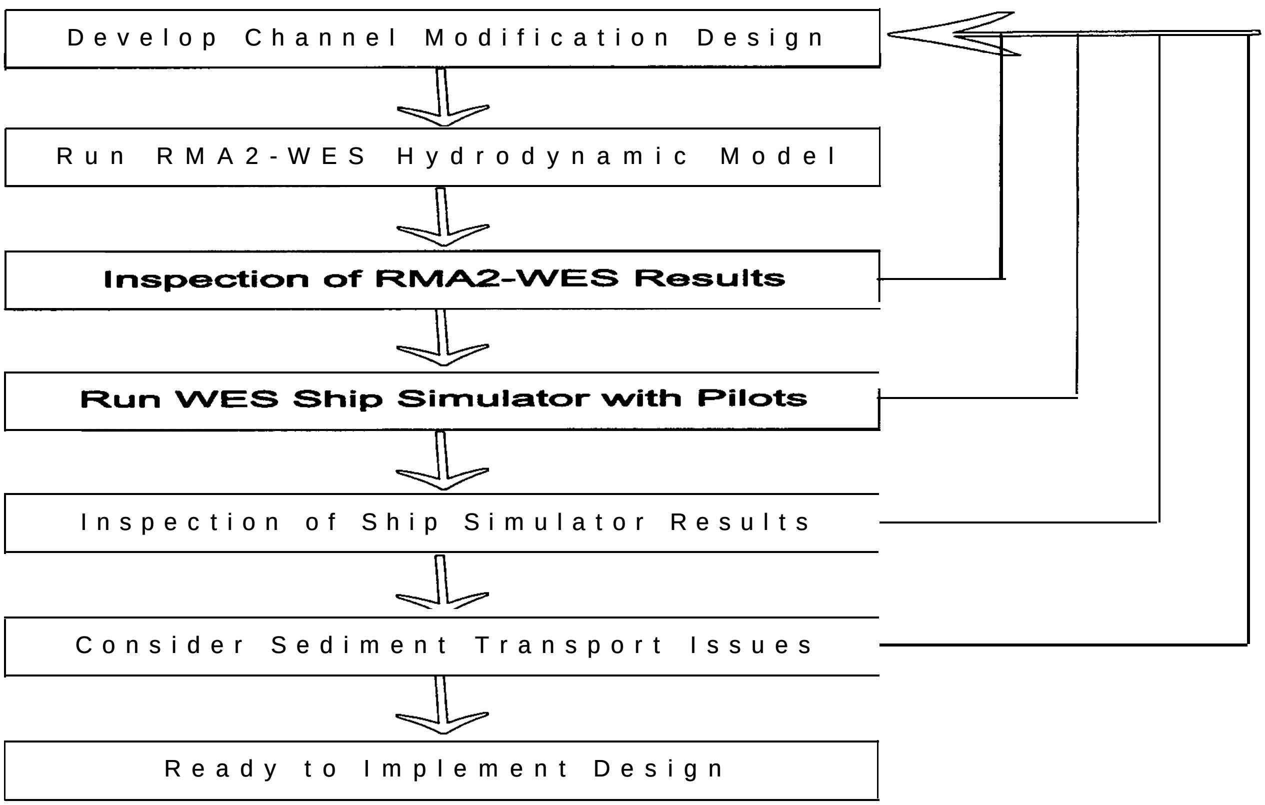

Figure 15-4. Typical events and feedback loops involved in WES ship simulator study

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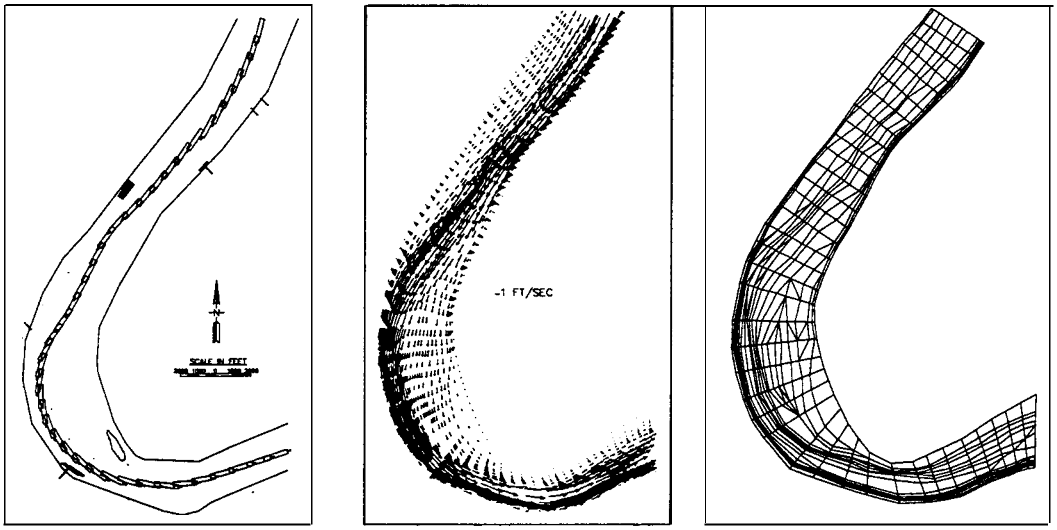

a. WES tow simulator track plots **b. RMA2-WES hydrodynamic solution** **c. Numerical model computational mesh**

Figure 15-5. Redeye Crossing of the Lower Mississippi River

a. RM42- WES governing equations.

(1) The generalized computer program RMA2-WES solves the depth-integrated equations of fluid mass and momentum conservation in two horizontal directions. The forms of the solved equations are

$$
\begin{aligned}
& h\frac{\partial u}{\partial t} + hu\,\frac{\partial u}{\partial x} + hv\,\frac{\partial u}{\partial y} \\
& - \frac{h}{\rho}\left(E_{xx}\,\frac{\partial^2 u}{\partial x^2} + E_{xy}\,\frac{\partial^2 u}{\partial y^2}\right) \\
& + gh\left(\frac{\partial a}{\partial x} + \frac{\partial h}{\partial x}\right) \\
& + \frac{gun^2}{(1.486h^{1/6})^2}\left(u^2 + v^2\right)^{1/2} \\
& - \zeta V_a^2 \cos\psi - 2h\omega v \sin\phi = 0
\end{aligned}
\qquad (15\text{-}1)
$$

$$
\begin{aligned}
& h\,\frac{\partial v}{\partial t} + hu\,\frac{\partial v}{\partial x} + hv\,\frac{\partial v}{\partial y} \\
& - \frac{h}{\rho}\left(E_{yx}\,\frac{\partial^2 v}{\partial x^2} + E_{yy}\,\frac{\partial^2 v}{\partial y^2}\right) \\
& + gh\left(\frac{\partial a}{\partial y} + \frac{\partial h}{\partial y}\right) \\
& + \frac{gvn^2}{(1.486h^{1/6})^2}\left(u^2 + v^2\right)^{1/2} \\
& - \zeta V_a^2 \sin\psi + 2\omega hu \sin\phi
\end{aligned}
\qquad (15\text{-}2)
$$

$$
\frac{\partial h}{\partial t} + h\left(\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y}\right) + u\frac{\partial h}{\partial x} + v\frac{\partial h}{\partial y} = 0 \qquad (15\text{-}3)
$$

*

* where

h = depth

u,v = velocities in the Cartesian directions

x,y,t = Cartesian coordinates and time

ρ = density of fluid

E = eddy viscosity coefficient, for xx = normal direction on x-axis surface, for yy = normal direction on y-axis surface, for xy and yx = shear direction on each surface

g = acceleration due to gravity

a = elevation of bottom

n = Manning's roughness n-value

1.486 = conversion from SI (metric) to non-SI units

ζ = empirical wind shear coefficient

V_a = wind speed

ψ = wind direction

ω = rate of earth's angular rotation

ϕ = local latitude, Coriolis

(2) Equations 15-1 through 15-3 are solved by the finite element method using the Galerkin method of weighted residuals. The elements may be 1-D lines, or 2-D quadrilaterals or triangles, and may have curved (parabolic) sides. The shape functions are quadratic for velocity and linear for depth. Integration in space is performed by Gaussian integration. Derivatives in time are replaced by a nonlinear finite difference approximation. Variables are assumed to vary over each time interval in the form

$$f(t) = f(0) + at + bt^c \qquad t_0 \le t \le t_0 + \Delta t \qquad (15\text{-}4)$$

which is differentiated with respect to time, and cast in finite difference form. Letters a, b, and c are constants. It has been found by experiment that the best value for c is 1.5 (Norton and King 1977).

(3) The solution is fully implicit and the set of simultaneous equations is solved by Newton-Raphson nonlinear iteration. The computer code executes the solution by means of a front-type solver, which assembles a portion of the matrix aud solves it before assembling the next portion of the matrix.

(4) RMA2-WES is based on the earlier versions (Norton and King 1977) but differs in several ways. It is formulated in terms of velocity (v) instead of unit discharge (vh), which improves **some aspects** of the code's behavior. Other differences from the earlier versions include the following:

(a) Employs new numerical solution algorithms.

(b) Permits wetting and drying of areas within the mesh.

(c) Permits wetlands to be simulated as either totally wet/dry or as gradually changing wet/dry states.

(d) Permits specification of turbulent coefficients in directions other than along the x- and z-axes.

(e) Accommodates the specifications of hydraulic control structures in the network.

(f) Permits the use of automatic assignment of friction and turbulent coefficients.

(g) Permits input in either non-SI or SI units.

(5) Additionally, the following have been incorporated into the RMA2-WES model as a result of deep- and shallow-draft research and applications.

(a) Incorporated a secondary flow ('bendway") corrector.

(b) Improved the RMA2-WES documentation and provided resolution guidelines.

(c) Provided an on-line point-and-click documentation capability on the PC.

(d) Incorporated a documentation icon within the graphical user interface on the PC.

*

b. The principle of bendway correction.

(1) The secondary flow (or "bendway") corrector was added to the RMA2-WES model. The modified program, designated as version 4.35, solves a transport equation for streamwise vorticity and converts it to accelerations due to secondary currents. These additional accelerations result in improved predictions of the traditional depth-averaged velocity calculations. Their effect is to reduce velocities on the inside of river bends and increase them on the outside of bends. The modeler may activate or deactivate the secondary flow corrector as required for its application. This enhancement permits RMA2-WES to be successfully used for some study areas that otherwise would have required the 3-D model.

(2) The theoretical basis of the bendway correction was developed for the depth-averaged finite difference numerical model, STREMB (Bernard and Schneider 1992).

(3) The bendway correction is accomplished by first solving an additional equation for the transport of streamwise vorticity. Vorticity is a measure of rotation of flow. Streamwise vorticity at a point is equal to the velocity of the fluid about the axis in the streamwise direction of flow. Streamwise vorticity is in the vertical plane perpendicular to the direction of flow and is related to the radial accelerations that cause the helical flow pattern.

(4) The transport equation for streamwise vorticity is

$$\frac{\partial\Omega}{\partial t} + \frac{\partial\Omega}{\partial x} + \frac{\partial\Omega}{\partial y} = \frac{A_S\sqrt{C_f}\,|u|^2}{Rh\left(1 + 9h^2 / R^2\right)} - D_S\sqrt{C_f}\,\Omega\frac{|u|}{h} + \frac{1}{h}\nabla\,(vh\nabla\Omega) \qquad (15\text{-}5)$$

where

Ω = streamwise vorticity

$A_S = 5.0$

C = friction coefficient

$|u|$ = magnitude of the velocity vector

R = local radius of curvature

$D_S = 0.5$

Units of vorticity are sec^{-1}.

(5) The additional shear stress caused by the secondary, helical flow is calculated from streamwise vorticity at each node. The components of this shear stress are added to the other terms (friction, slope, Coriolis) in the governing equations.

15-9. RMA2-WES Documentation

With the technological advancements of the computer industry and the evolution of computational algorithms, it was evident that published documentation could be quickly outdated. To address the evolution of the "art" of numerical modeling, a living approach to documentation was selected. The RMA2-WES ***"DOC-TO-HELP"*** hypertext documentation is regularly updated and available for download from the World Wide Web (WWW). After downloading it to your PC, you may view the on-line documentation on any PC running windows. The WWW address for the documentation:

http://hlnet.wes.army.mil/software/tabs/tabs.htm

15-10. Graphical User interface

All USACE employees performing surface water analyses for the USACE may obtain a copy of the Graphical User Interface developed by Brigham Young University (BYU). Two generations of graphical interfaces are compatible with TABS-MD: FastTABS (1989-1994) and SMS (1995-present). To obtain a copy of the SMS interface, download the proper executable for your computer site and complete the request form available from the WWW at this address:

http://hlnet.wes.army.mil/software/interfaces/sms/smsreg.htm

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Appendix A
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APPENDIX B

CHECKLIST FOR STUDIES REQUIRED

The development of waterways for navigation involves the study and evaluation of many factors to assure efficiency, safe conditions, and reliability at minimum cost. The following are some of the studies and factors that should be considered in the planning and design phase:

a. Hydraulic and geological characteristics of the stream, and locations of existing bridges, highways, railroads, and industrial complexes.

b. Channel depths and widths available and requirements for traffic anticipated.

c. Need for channel realignment, training structures, and/or locks and dams.

d. Optimum locations for locks and dams, if required.

e. Alignment and velocity of currents and movement of sediment in critical reaches and at proposed lock and dam sites.

f. Effects of various arrangements of lock or locks, dam and overflow weirs, and auxiliary lock walls including new concepts and orientation of the structures.

g. Number and size of spillway gate bays and effects of overflow weirs and embankment on cost of project and on navigation conditions.

h. Use of a navigable pass to reduce the height of lock walls.

i. Effects of structures on flooding, overbank flow, and movement of sediment.

j. Effects of various types of lock filling and emptying systems on navigation.

k. Effects of developments on water quality and local environment.

l. Feasibility of powerhouse installation and effects on navigation.

m. Feasibility of water conservation methods.

n. Effectiveness of various types of river training and stabilization structures and navigation aids.

o. Navigation and flow conditions during construction.

p. Hydraulic model studies to determine:

(1) Optimum design for spillway and stilling basin operating under various conditions (full or partial width).

(2) Navigation conditions in lock approaches, best arrangement of locks, dam, and training structures, movement of ice and debris, and conditions during construction (comprehensive fixed bed).

(3) Effects of structures on movement of sediment, channel development in lock approaches and in critical reaches, and scour with various cofferdam plans for construction (comprehensive movable bed).

(4) Conditions at other locations as needed such as harbor entrances, docking and assembly areas, and at bridges (fixed or movable bed).

q. Baseline environmental and water quality data collection and evaluation, and consideration of applicable environmental laws and regulations.

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