DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

CECW-EH

Pamphlet
No. 1110-2-14

30 November 1997

Engineering and Design
MONOGRAPH PREPARED BY M. PETERSEN ON INLAND NAVIGATION AND CANALIZATION

1. Purpose

This pamphlet disseminates a monograph compiled by Margaret Sara Petersen, Emerita Associate Professor, Department of Civil Engineering and Engineering Mechanics, the University of Arizona.

2. Applicability

This pamphlet applies to all USACE commands with responsibility for hydraulic and structural design, construction, and operation of inland navigation projects.

3. General

The monograph was originally prepared for use by the Chinese as a goodwill gesture, and not as a contract. The author used her knowledge and input from various sources within USACE. She expressed an interest in sharing her work at no cost to USACE.

4. Distribution Statement

Approved for public release; distribution is unlimited.

5. Author

Margaret Sara Petersen is a highly respected professional civil engineer, teacher, and author. She worked as a hydraulic engineer with USACE for 30 years ending as Chief, Planning Branch, Sacramento District, before becoming an associate professor. Her work with the USACE encompassed some of the nation's largest water resource development projects including the Mississippi River flood-control and navigation effort, Missouri River reservoirs and navigation improvements, Arkansas River navigation improvements, California's Sacramento-San Joaquin Delta project, and the $1 billion Marysville Lake reservoir and Yuba River in California. After leaving USACE, Petersen developed new graduate-level courses, wrote two textbooks, and lectured in China, Morocco, and South Africa.

FOR THE COMMANDER:

Otis Williams

OTIS WILLIAMS
Colonel, Corps of Engineers
Chief of Staff

INLAND NAVIGATION AND CANALIZATION

Margaret S. Petersen, Emerita Associate Professor
Department of Civil Engineering and Engineering Mechanics
The University of Arizona, Tucson, Arizona, U.S.A.

TABLE OF CONTENTS

			Page
1.	**INTRODUCTION**		1-1
2.	**PLANNING FOR CANALIZATION**		2-1
3.	**NAVIGATION LOCKS AND DAMS**		3-1
4.	**PHYSICAL FACTORS AFFECTING SITING OF NAVIGATION STRUCTURES**		4-1
4.1	-	Terrain	
4.2	-	Geology and Soils	4-1
4.3	-	Streamflow and River Stage	4-1
4.4	-	Ground Water	4-2
4.5	-	Climate	4-2
4.6	-	Sediment	4-2
4.7	-	Environmental Resources	4-3
4.8	-	Infrastructure and Commercial Resources	4-3
5.	**INLAND NAVIGATION CRITERIA**		5-1
5.1	-	Minimum Dependable Depth.	5-1
5.2	-	Adequate Channel Width	5-2
5.3	-	Freedom from Hazardous Currents	5-3
5.4	-	Minimizing Lock Transit Time	5-3
5.5	-	Terminal Facilities	5-4
6.	**OTHER DESIGN REQUIREMENTS**		6-1
6.1	-	Flood Stages	6-1
6.2	-	Drainage	6-1
6.3	-	Water Supply Intakes	6-1
6.4	-	Sewage Contamination	6-2
6.5	-	Vector Control	6-2
6.6	-	Fish and Wildlife	6-2
6.7	-	Recreation	6-3
6.8	-	Hydropower	6-3

7. **NAVIGATION DAMS** 7-1

7.1 - Navigable Movable Dams 7-1
7.2 - Spillways 7-2
7.3 - Spillway Gates 7-3
7.4 - Spillway Piers 7-5
7.5 - Ice 7-5

8. **NAVIGATION LOCKS** 8-1

8.1 - General Considerations 8-1
- Lock Location 8-1
- Cofferdams for Construction 8-2
- Access to Construction Site 8-3
- Availability of Construction Materials 8-3
8.2 - Lock Design Criteria 8-3
- Lock Size and Number of Locks 8-3
- Lock Lift 8-4
8.3 - Lock Types 8-4
8.4 - Lock Depth and Lock Floor 8-5
8.5 - Lock Gates and Sills 8.6
- Miter Gates 8-8
- Tainter Gates 8-8
- Sector Gates 8-8
- Lift Gates 8-8
- Emergency Closure Sills 8-8
8.6 - Lock Walls 8-9
- Lock Chamber Walls 8-10
- Upper and Lower Gate-Bay Walls 8-10
- Culvert-Intake Walls 8-10
- Culvert-Discharge Walls 8-11
- Approach (Guard and Guide) Walls 8-11
8.7 - Lock Filling and Emptying Systems 8-12
- Hawser Stresses 8-13
- Filling and Emptying Over, Between, or Around Lock Gates . 8-14
- Filling and Emptying by Valves in Gates or through Short Culverts 8-14
- Filling and Emptying through Wall Culverts and Ports or Laterals 8-15
  - Early conventional wall culvert and port systems . . 8-15
  - Modern systems 8-15
  - Intake manifolds 8-16
  - Control valves 8-18
  - Wall culverts 8-19
  - Wall-port systems 8-19
  - TVA multiport system 8-20

- Lock chamber lateral diffusers 8-20
- Lock emptying systems 8-21
- Bottom Longitudinal Filling and Emptying Systems . . 8-22
- Culvert area ratios 8-23
- Longitudinal floor distribution culverts . . . 8-23
- Port manifolds 8-23
- Ports 8-23
- Operation 8-24

8.8 - Emergency Closure Facilities for Locks 8-24

9. **NAVIGATION CONDITIONS IN LOCK APPROACHES** 9-1

9.1 - Dardanelle Lock and Dam, Arkansas River 9-2
9.2 - Lock and Dam 2, Red River 9-2
9.3 - Robert S. Kerr Lock and Dam, Arkansas River 9-3
9.4 - Olmsted Lock and Dam, Ohio River 9-4
9.5 - Canal Surge and Tow Squat 9-4
- Temporary Lock 52, Ohio River 9-6
- Bay Springs Lock and Dam 9-6
- Lock and Dam 17, Verdigris River (Choteau Lock and Dam, Arkansas River Navigation Project). . . . 9-8
- Shoaling 9-9

10. **DREDGING** 10-1

10.1 - Arkansas River Dredging 10-2
10.2 - Mississippi River Dredging 10-5
10.3 - Missouri River Dredging 10-6
10.4 - Red River Dredging 10-6
10.5 - Effect of Hinged Pool Operation on Maintenance Dredging . . 10-7
10.6 - Dredging Equipment 10-7
- Mechanical Dredges 10-7
- Hydraulic Suction Dredges 10-8
- Hopper dredges 10-8
- Sidecasting dredges 10-8
- Hydraulic pipeline dredges 10-8

10.7 - Dredged Material Disposal 10-9

11. **INNOVATIVE LOCK DESIGN** 11-1

11.1 - Need for Rehabilitation or Replacement of Navigation Structures . . 11-1
11.2 - Increasing Lock Capacity 11-2
11.3 - Need for Innovation in Lock Design 11-3
11.4 - Innovative Lock Design Program 11-3
- Winfield Locks and Dam, Kanawha River 11-4
- Modeling Program 11-4

- Marmet Lock and Dam, Kanawha River 11-4

APPENDIX A **REFERENCES** A-1

APPENDIX B **RED RIVER WATERWAY** B.1

B.1 - Project Description B-1
B.2 - Sediment B-2
B.3 - Hinged Pool Operation B-3
B.4 - Reaeration B-4
B.5 - Optimization of Spillway Design B-4
Attachment B.1 - Typical Spillway Optimization Study, Red River, Louisiana, Appendix D, CE EM 1110-2-1605, 12 May 1987) . . . B-10

INDEX I-1

CONVERSION FACTORS, NON-SI TO SI UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	by	To obtain
inch	2.54	centimeters
foot	0.3048	meters
mile	1.609	kilometers
square feet	0.0929	square meters
acre	4,047	square meters
acre	0.405	hectare
square mile	259	hectare
square mile	2590	square kilometer
cubic feet	0.0283	cubic meters
cubic yards	0.765	cubic meters
acre-feet	1,233	cubic meters
cubic feet per second	0.0283	cubic meters per second
feet per second	0.305	meters per second
pounds (mass) pounds per cubic foot	0.454 16.0185	kilograms kilograms per cubic meter
pounds (force) pounds per square inch	0.454 6,895	kilograms pascals
gravitational acceleration (ft/sec^2)	9.807	meters/sec^2

FOREWORD

This monograph describes the design of inland waterways in the United States by the U.S. Army, Corps of Engineers (USACE). The Corps of Engineers maintains an 11,000-mile shallow-draft inland waterways system with 211 locks at 168 sites and more than 180 navigation dams with normal heads ranging from one ft to over 100 ft.

Sections 7 and 8 summarize criteria generally used for design of canalization projects constructed in the United States in the period 1950 through 1993. Material presented draws extensively on guidelines of the U.S. Army, Corps of Engineers; Miscellaneous Papers by T. E. Murphy and J. P. Davis issued by the USACE Waterways Experiment Station, Vicksburg, Mississippi; and on the book *River Engineering*, M. S. Petersen, Prentice Hall, 1986. Many of the existing navigation structures have been in use for more than 50 to 60 years and now are of insufficient size or have deteriorated to the point where they cannot meet the needs of the shipping industry and, accordingly, are in need of rehabilitation or replacement.

The Water Resources Development Act of 1986, which authorized replacement of eight locks, also introduced requirements for cost-sharing for new locks and for major rehabilitation of existing locks. The cost of construction is now divided equally between the federal government and the Inland Waterway Trust Fund (IWTF). Funds accrue to the IWTF from taxes on fuel used on the inland waterways system, currently 20 cents per gallon.

In recent years the cost of replacement structures has increased significantly, and the Corps of Engineers is exploring innovative modifications of traditional lock designs to lower construction and operation and maintenance costs while meeting the needs of today's navigation industry. Currently available information on these innovative measures is presented in Section 11.

Appreciation is expressed to Samuel B. Powell, Office of the Chief of Engineers; Tasso Schmidgall, Hydraulics Section, Southwestern Division, Corps of Engineers; Gary Dyhouse, St. Louis District, Corps of Engineers; and John George, Hydraulic Structures Division, USACE, Waterways Experiment Station, for their kind contributions to this work.

INLAND NAVIGATION AND CANALIZATION

Margaret S. Petersen, Emerita Associate Professor
Department of Civil Engineering and Engineering Mechanics
The University of Arizona, Tucson, Arizona, U.S.A.

1 INTRODUCTION

This monograph describes the design of shallow-draft inland waterways in the United States (U. S.) by the U.S. Army, Corps of Engineers (USACE). The Corps maintains an 11,000-mile inland waterways system, Figure 1.1, with 211 locks at 168 sites and more than 180 navigation dams with normal heads from one ft to over 100 ft. This system handled more than 530 million tons of commerce in 1992, and total ton-miles that year was a record 271 billion (Antle and Grier, 1995).

Commodities moving on the system vary geographically and include coal, grain, petroleum, chemicals, and aggregates, all relatively low-value bulk materials. In the order of 50 percent of U.S. grain exports and 20 percent of coal exports move on the system. About 60 percent of electricity generated in the U.S. is coal-fired, and about 25 percent of this coal is transported by water (Antle and Grier, 1995).

Most inland navigation facilities in the U.S. are about 50 years old, and systems on some rivers, modernized in the past, are in need of rehabilitation or replacement at this time. Navigation locks and dams on the Upper Mississippi and Illinois Rivers were constructed in the 1930s, those on the Tennessee River in the 1950s and 60s, and those on the Ohio, Arkansas, and Columbia/Snake Rivers in the 1960s and 70s. The most recently completed systems are the Tennessee-Tombigbee Waterway opened in 1985 and the Red River Navigation Project, Louisiana, opened in December 1994. Modernization and replacement of older locks is continuing.

National objectives in developing rivers for safe and efficient inland navigation include:

a. Promoting the production and distribution of food resources.
b. Promoting the expansion of existing and development of new industrial production.
c. Enhancing economic development in general.
d. Enhancing social well-being.
e. Achieving these objectives while preserving and enhancing fish and wildlife resources and environmental quality.

Development of inland waterways for commercial navigation can be achieved in three general ways:

a. "Open river" development of rivers that have adequate flows to provide navigable depths in the navigation season.

b. "Canalized" development of rivers that do not have sufficient depths for navigation by a series of locks and dams to impound pools of adequate depth.

c. Canal development by excavating channels across land areas.

The type of development to be used on a specific river depends on local conditions and on costs if more than one type of development would be equally suitable. The primary consideration is whether or not flows will be adequate to provide sufficient depth in the "navigation season."

Local climate may limit the navigation season to periods of adequate rainfall or to warmer months in cold climates where ice blocks the river in winter. Also, high river stages and high velocities during floods interrupt navigation. The Upper Mississippi River freezes over every winter, and the river is closed to navigation from about early December to mid-March due to ice. The Upper Mississippi is also closed to navigation at other times of year during floods when the dams "go out of operation," (all spillway gates fully open) and pool levels are within 2 ft of the top of lock walls. At Lock and Dam 22, for example, this is a flow of 160,000. Navigation ceases on both the Arkansas River and the Red River at the 10 percent recurrence frequency when velocities and currents become too high for safe and efficient tow operation.

Few rivers, except in tidal reaches, have adequate dimensions and suitable velocities for open river navigation. Where streamflow does not naturally provide adequate depths for open-river development throughout the year, upstream reservoir storage may be used to provide controlled releases and adequate depths in downstream reaches. Depths can be increased also by stabilization and rectification work and by maintenance dredging, and levees may be used to confine flows to a designated floodway.

Canalization (systems of locks) is used to provide adequate depth for navigation in streams having little discharge and, therefore, depths too shallow for navigation; in a waterway having a steep slope and velocities too high for navigation; at a waterfall or rapids in a stream that otherwise provides adequate depth in other reaches.

Canals cut through land generally are used to connect two bodies of water and to bypass rock outcrops and rapids in rivers. Canals are expensive, requiring acquisition of large tracts of land for the canal and for disposal of excavated material, and canal banks often require protection from wave damage because of the restricted channel width.

Development of safe and efficient inland navigation is based on providing the following:

a. Channels of adequate dimensions (depth and width) for navigation.

b. Safe streamflow velocities that are not a hazard to navigation traffic.

c. Harbors and related appurtenances for receipt and shipment of commodities.

d. Compatibility of navigation requirments with other developments, including flood protection works, transportation networks (roads, railroads), and utility crossings.

Navigation projects typically include such basic components as:

a. Spillway (gated, uncontrolled, or wickets).
b. Overflow weir or embankment.
c. Non-overflow embankment.
d. Locks.
e. Navigable pass.
f. Outlet works.
g. Water quality enhancement facilities.
h. Fish passage facilities.
i. Aids to navigation.

Additionally, facilities for generation of hydroelectric power, releases for irrigation or stream maintenance, and recreation may be included, depending on local conditions.

Planning, design criteria, and operating procedures for navigation projects should consider measures to avoid or minimize adverse ecological impacts, mitigate adverse effects, and provide environmental enhancement. Particular concerns in the United States (U.S.) are to improve low dissolved oxygen levels downstream of dams by flow aeration and to prevent nitrogen supersaturation on high spillways.

Figure 1.1. Shallow-draft Inland Waterways System, United States.

2 PLANNING FOR CANALIZATION

The basic objective in developing a design for a canalized waterway is to provide facilities to meet projected future shipping needs in the most economical way consistent with protection and enhancement of social and environmental resources throughout the useful life of a project, (project life). Project life for navigation work in the United States is usually taken as 50 years.

Investigations to determine if a project can handle projected future transportation equipment and tonnage efficiently and safely include:

a. Economic studies relating to the amount and type of traffic that would use the new waterway, including:

- Projections of commodities that would move on the new waterway. What commodities would move? In what amount (annual tonnage)? From what origin, and to what destination? In what season would they move? Is there return traffic?
- Estimates of transportation benefits (savings) and intangible effects related to use of the waterway.
- Estimates of effects of the project on economic development of the region.

b. Evaluation of existing streams, including:

- Flood magnitude and frequency.
- Channel widths and depths at different seasons of year.
- Channel radii in bends at different seasons of year.
- Water quality.
- Sediment load.
- Bank erosion.
- Existing transportation facilities.
- Existing and planned river crossings (highways, railroads, pipelines, power lines).
- Existing and planned industrial development.
- Existing and planned port facilities.
- Important habitat areas and other environmental resources.

c. Evaluation of navigation equipment.

- Type, size, and draft of navigation equipment (towboats, barges, vessels) currently using the waterway or connecting channels.
- Projected types and size of equipment likely to use the waterway in the future.

d. Physical constraints on a canalization project.

- Are there any geographic or geological features along the river that are likely to make canalization clearly infeasible?

- Is streamflow augmentation needed? Is it feasible? (Are there upstream reservoir sites that can be developed for storage and low-flow augmentation?)
- Is there need for rectification and stabilization of the river to develop adequate navigable depths and widths?

e. What is optimum lock size for projected traffic and navigation equipment? Number of lock transits required annually throughout the project life to meet needs of shippers?

f. Is a single lock, or multiple locks, most economically efficient for handling projected traffic at each lock site?

The views of towboat captains who will use the waterway and the U.S. Coast Guard are requested with regard to channel dimensions and lock layout. The U.S. Coast Guard is responsible for navigation safety and navigation aids, such as channel lights and marking buoys, on inland waterways in the United States.

Size of tows using the inland waterways in the U. S. varies widely. Representative tow sizes for some waterways are summarized as follows:

a. Mississippi River.

- Upper Mississippi River (canalized). Standard tow size is 15 barges, in a configuration three barges wide and five barges long, Figures 2.1 and 2.2. Towboats have 3200 to 6000 horsepower.
- Middle Mississippi River (open-river). For downbound traffic, standard tow size is 25 barges, in a configuration five barges wide and five barges long. For upbound traffic, standard tow size is 30 barges, in a configuration five barges wide and six barges long. Towboats have 5600 to 6000 horsepower.
- Lower Mississippi River (open-river). For downbound traffic, standard tow size is 30 to 35 loaded barges. For upbound traffic, tow size ranges from 30 to 45 barges depending on river conditions and the mix of loaded and empty barges in the tow, Figure 2.3. Towboats have from 5600 to 10,500 horsepower.

b. Arkansas River (canalized). Standard tow size is eight barges, in a configuration three barges and three barges long, with the towboat occupying the middle slot in the last row of barges. Maximum tow size is 17 barges in a three barge wide by six barge configuration, with the towboat occupying the middle slot in the last row. Overall tow length is limited to 1200 ft because of the tight (small) radii of some bends.

c. Missouri River (open-river). Above Kansas City, standard tow size is three or four barges, in a configuration two barges wide and two long, and maximum tow size is six barges, in a configuration two barges wide and three long. Below Kansas City, standard tow size is six to nine loaded barges and 12 empty barges.

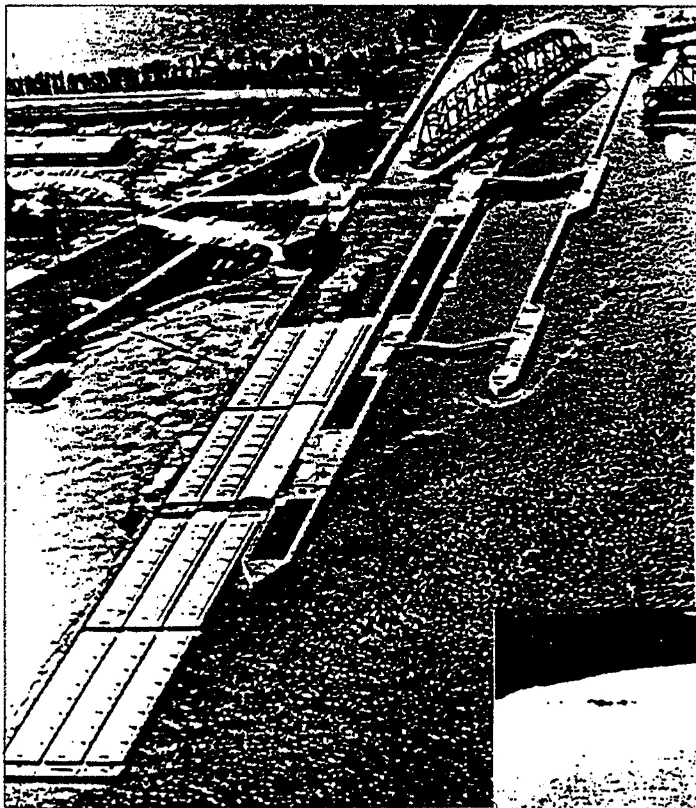


Figure 2.1. Twelve-barge tow reassembled after double lockage, Lock and Dam 15, Upper Mississippi River. (Rock Island *Argus*)

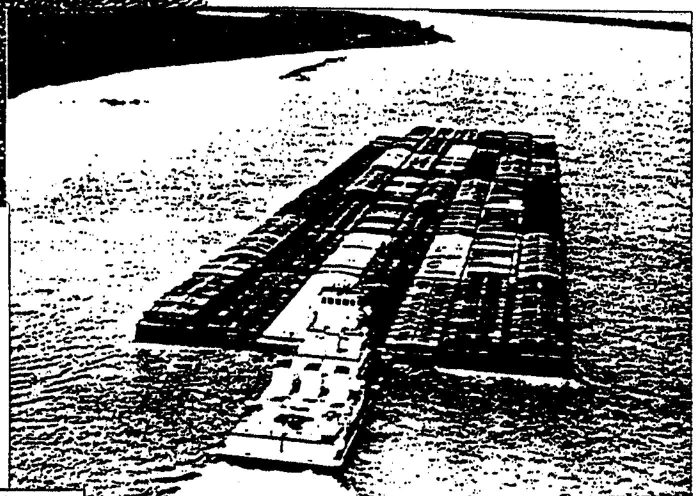


Figure 2.2. Forty-eight-barge tow, Lower Mississippi River (U.S. Army, Corps of Engineers, Vicksburg District)

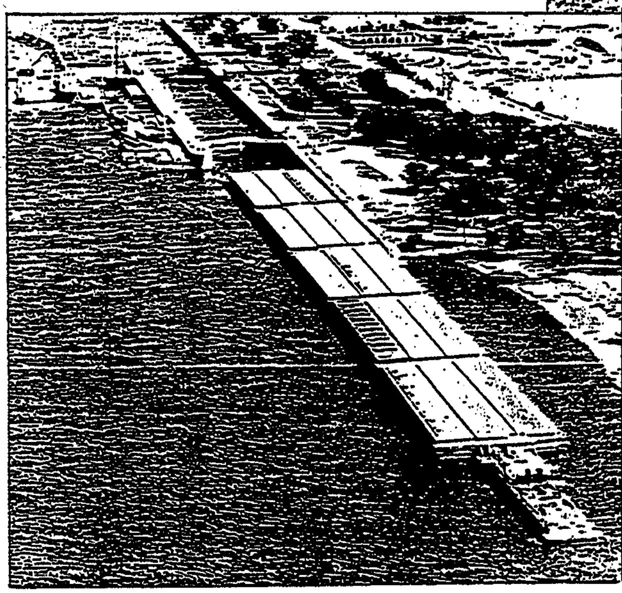


Figure 2.3. Fifteen-barge tow approaching Lock and Dam 22, Upper Mississippi River (U.S. Army, Corps of Engineers, Rock Island District,)

3 NAVIGATION LOCKS AND DAMS

A canalized river is a river that has been transformed from a free-flowing stream to a series of "slackwater" pools with low flow velocities by a series of locks and dams along the stream. Navigation dams impound the pools, and the locks make it possible for vessels to pass through the dams, either upstream or downstream, from one pool level to the next, Figure 3.1. "Low-head" dams are dams with heads of 10 to 40 ft, and "high-head" dams have heads in excess of 40 ft. Lock with lifts of less than 30 ft are classified as low-lift locks; with lifts of 30 to 50 ft as medium-lift; and with lifts of more than 50 ft as high-lift locks.

Principal criteria for selection of sites for navigation locks and dams in a particular reach are related to physical characteristics of the reach (foundation conditions, current directions and magnitude, sediment transport); local drainage conditions; stability of the channel bed at the site; urban, industrial, and agricultural development; transportation infrastructure; and environmental resources, as follows:

a. Reach conditions.

- A history of relatively permanent banks in the reach because recently formed banks are usually low and costly to protect.
- A channel alignment that provides fairly straight approaches to the lock, without a sharp bend upstream or a crossing downstream near the lock, to minimize cross currents in the lock approaches.
- Sufficient width of main channel to accomodate the required spillway length and the lock, but not excessively wide and costly, or so narrow as to require extensive bank excavation. As lock sites are frequently on the deep concave side of the channel with the lock set out from the bank to provide adequate approach alignment, space left in the main channel for the spillway may be materially reduced.
- A high narrow overbank that eliminates the need for embankments to impound the normal pool and reduces embankment heights required for roads to provide land access to the lock. A narrow overbank also tends to concentrate flood flows in the main channel, tending to maintain a deeper channel downstream of the dam in the head of the next pool where adequate navigation depth is critical.

b. Drainage. Insofar as conditions permit, navigation locks and dams should be sited so that principal tributaries and drains enter the channel near the head of a navigation pool, rather than in the lower part of the pool, to avoid interference with drainage.

c. Channel bed. The elevation of the future stable bed of the stream must be estimated, taking into account the effects of any cutoffs, any reduction in sediment load due to upstream storage reservoirs, the effects of any channel contraction works, and the backwater effects of the navigation dams.

Most locks on inland waterways in the United States are 110 ft wide by 600 or 1200 ft long, with gates at both ends (at the upper pool and at the lower pool). There are water passages in the lock walls, floor, gate sills, or in the gates themselves to admit water to the lock chamber from the upper pool to fill the lock and to discharge water from the lock chamber to the lower pool to empty the lock, as illustrated schematically in Figure 3.2.

For a vessel to proceed *downstream* through a lock, the lock is operated in the following sequence:

a. The emptying valves and lower and upper lock gates are closed.
b. The filling valves are opened to fill the lock and raise the water surface in the lock chamber to the same elevation as the upper pool.
c. The upper lock gates are opened, and the vessel moves into the lock chamber.
d. The upper lock gates and filling valves are closed.
e. The emptying valves are opened to lower the water surface in the lock (and the vessel) down to the level of the lower pool.
f. The lower lock gates are opened.
g. The vessel moves out of the lock chamber and into the lower pool.

Locks are sized for a design vessel or design tow (a towboat and barges), usually those in use on the waterway, or adjoining waterways, at the time. However, if changes in equipment size can be anticipated in the future with the project, such changes should be given due consideration in selecting lock chamber size. In the United States dimensions of barges and towboats have changed little over the years, but the number of barges in a tow has increased as towboat engine horsepower has increased.

Lock size affects the economic success of a waterway. If the locks are too small, traffic may not develop as projected because of traffic delays in passing through the locks. If the locks are too large, fixed and operating costs may be so large as to make the project uneconomical. Uniformity of lock size from one waterway to another linking waterway is desirable to permit through navigation. Careful consideration should be given to lock size and to the number of locks at a given site. Two smaller locks may be more efficient in passing tows that one large lock.

Most locks on the Upper Mississippi River, constructed in the 1930s, have lock chambers 110- by 600-ft, and many tows using the river today are too large to pass through the locks in a single lockage. The 15-barge tow at Lock and Dam 22, Canton, Missouri, shown in Figure 2.3, will require two lockages. The 12-barge tow exiting Lock and Dam 15, Rock Island, Illinois, has been reassembled after double lockage, Figure 2.1. Larger tows, such as the 48-barge tow shown in Figure 2.2, are common on the Lower Mississippi River where open-river conditions prevail, the channel is wide, and there are no locks.

"Lockage time," or "lock transit time," includes the time from when a tow or vessel begins to proceed into a lock, is locked through, and exits the lock to the point where an opposite-bound tow can enter the lock. Large tows must be slow and cautious when entering a lock because the water displaced flows out of the lock along the sides of and under the tow. Filling and emptying times for a lock are designed to be as short as possible without causing excessive turbulence, surges, or cross currents in the lock chamber that might damage the tow or cause the tow to damage the lock.

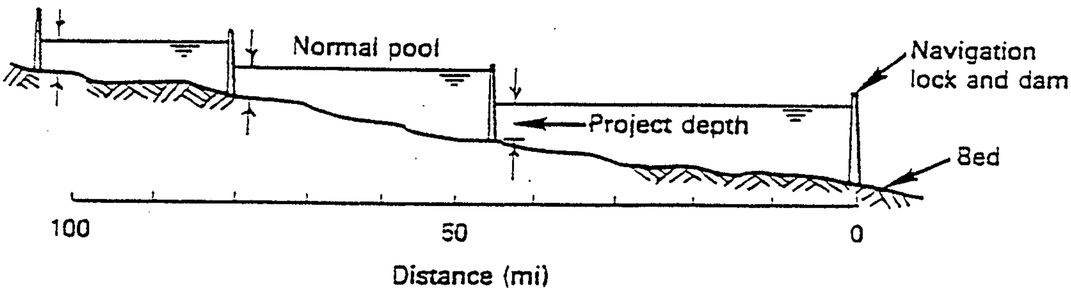


Figure 3.1 Spacing of navigation dams.

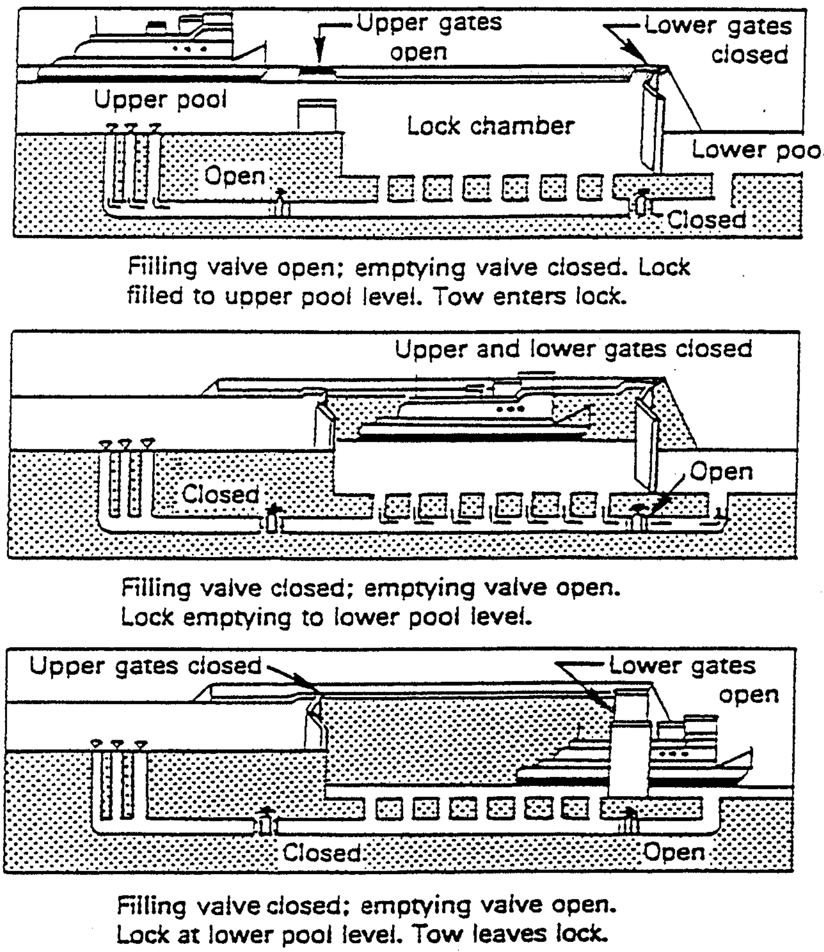


Figure 3.2 How locks operate.

4. PHYSICAL FACTORS AFFECTING SITING OF NAVIGATION STRUCTURES

Physical conditions affecting affecting selection of sites for navigation locks and dams include the following.

4.1 Terrain

Stream gradient influences the number of dams required and the height and spacing of dams. On a steep river, the pools will be deeper and shorter than on a river of flatter gradient. Bank heights limit the pool elevations (and dam heights) that can be used without permanently flooding lands outside the normal channel limits.

The location of tributary streams may influence dam location because of the effects of tributary flood flows on dam operation, deposition of sediment carried by the tributary in the quiet water of the navigation pool above the dam, backwater effects along the tributary related to impoundment, and sediment deposition in the downstream reach of the tributary that could increase local flood heights along the tributary. In general, a dam site immediately above a major tributary is better than a site immediately below the tributary.

The valley cross section should be wide enough for the locks and a spillway of adequate length to pass flood flows without raising water surface elevations substantially. On alluvial rivers, if the channel must be widened significantly in the vicinity of the project to accommodate the required spillway length, problems with sediment deposition are likely to occur in the vicinity of the structure.

4.2 Geology and Soils

The best foundation material for a lock and dam is sound rock at reasonable depth, but structures can be built successfully on other materials. Because geologic formations often vary radically along a river, moving a damsite a few kilometers upstream or downstream may result in safer and more economical foundation materials. Locks set on alluvial materials usually require a pile foundation for structural stability.

River banks in the vicinity of a damsite should be relatively stable and permanent. Recently formed banks are usually low and costly to protect. Leakage through the dam foundation may result in piping that threatens structural failure of the lock and dam, and impervious cutoff walls may be needed. Impervious clay blankets upstream of the dam may be used to prevent loss of water by seepage from the upper pool.

4.3 Streamflow and River Stage

The spillway of a navigation dam is designed to pass the selected maximum design discharge, typically a lesser and more frequent flow than used for the design of high dams.

Minimum streamflow must be sufficient to operate the locks and to meet other water requirements, such as leakage through the locks and dam, seepage from the pool and under the dam, evaporation from the pool, and any required consumptive uses. If minimum flows are too low to meet these requirements, special measures are needed to reduce seepage, recirculate lockage water, or supplement low flows.

If large or rapid fluctuations in streamflow are typical, frequent use of spillway gates will be required to maintain normal pool elevation.

Maximum water surface level determines the minimum height for gate piers on the spillway crest and the clearance required for overhead structures, such as a bridge across the dam. Piers must be high enough for fully-open gates to clear the maximum design water surface. Navigation dams are designed to have minimum effect on flood levels through the pool, and the backwater effect is generally limited to about one foot.

Minimum stage affects design of the stilling basin below the dam spillway. Minimum pool elevation determines the extent of lands permanently flooded and, therefore, acquired for a project. In a canalized river, the water surface in navigation pools is generally above natural low-water elevations, and minimum pool level is the major factor determining the impact of the project on the groundwater table and drainage of adjacent lands.

At the head of a pool, water surface levels fluctuate between normal pool elevation and flood stages much the same as under preproject, open-river conditions. Depending on dam height, stages just upstream of the dam may be permanently above natural flood levels.

4.4 Ground Water

Maintaining pool levels that are higher than pre-project normal low river stages will increase ground water levels in the vicinity.

4.5 Climate

The effects of humidity in areas of frequent or prolonged fog and the combination of heat and humidity in the tropics must be given special consideration, especially in design and maintenance of electrical machinery and the metal parts of structures.

Temperature range also may influence the type and design of operating machinery selected. Ice can be a problem in cold climates if there is winter navigation, and special measures may be required to limit icing on gates, trash racks, water intakes, and lock chamber walls. Even without winter navigation, navigation structures in cold climates are designed to pass large volumes of ice to avoid ice jams in the river.

4.6 Sediment

In a typical low-head navigation project, spillway gate sills are set very near river bed elevation, Figure 4.1, and spillway gates are operated to pass flood flows with a minimum of surcharge so that essentially open-river conditions prevail at high flows and the river can continue to pass its normal sediment load.

Dams must be spaced along a river so that project depth exists in the upstream (head) end of each navigation pool, Figure 4.2. On alluvial rivers, some maintenance dredging typically is required in such reaches, and frequently is necessary to contract the channel locally to maintain sediment transport capacity at the heads of pools, Figure 4.2.

4.8 Environmental Resources

In designing a navigation project, consideration must be given to potential impacts on water quality; flora, fish, and wildlife resources; historical, archaeological, and paleontological resources; and recreational opportunities.

4.9 Infrastructure and Commercial Resources

Urban development, highways, railroads, bridges, and pipeline and utility crossings may need to be relocated or modified to accommodate a canalization project. Urban areas may be affected by changes in flooding pattern, rise in ground water levels, and pool levels that interfere with sewer outfalls. Problems in urban areas can be minimized by locating navigation dams upstream (rather than downstream) of urban areas where feasible or by using several low dams through an urban area rather than one higher structure.

Where there is extensive agricultural development in the river valley or mining in the overbank, consideration should be given to two or more lower dams, rather than a single dam, to reduce costs for land acquisition, relocations, and damages.

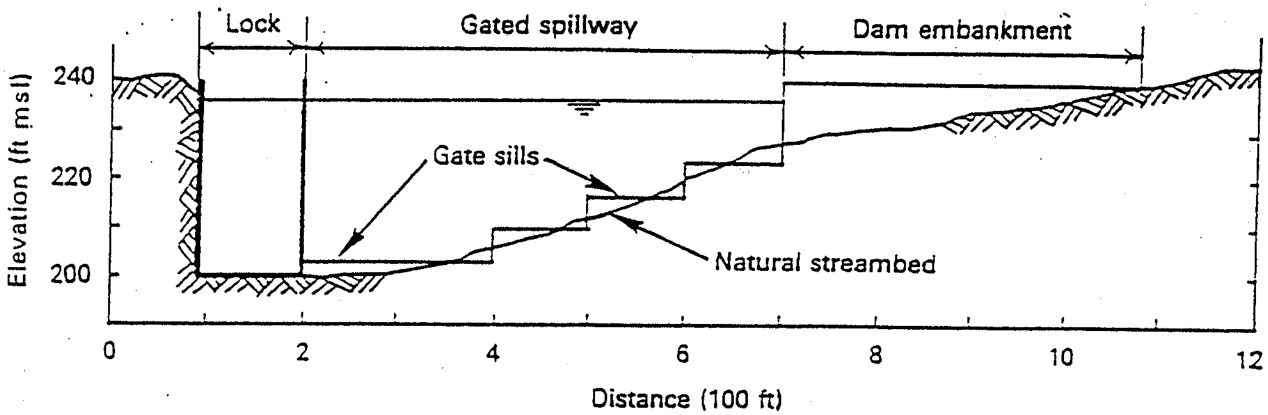


Figure 4.1 Cross section, typical low-head navigation dam.

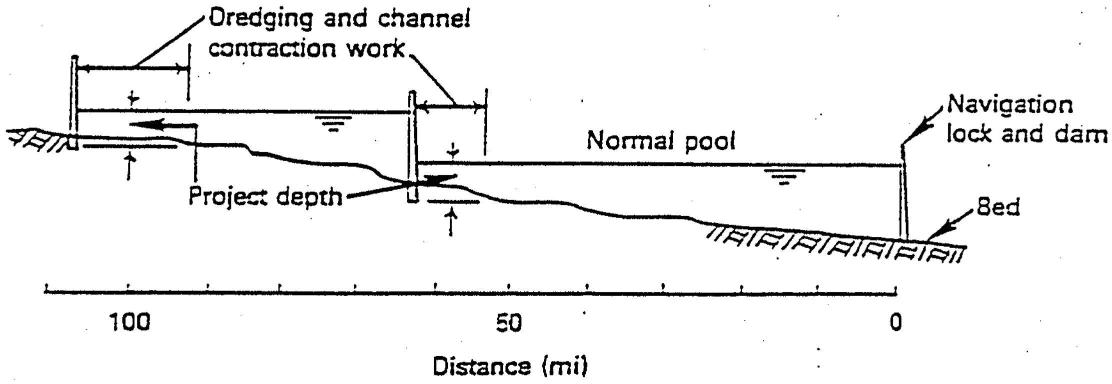


Figure 4.2 Typical location of dredging and contraction work at heads of pools.

5. INLAND NAVIGATION CRITERIA

General design requirements for inland navigation channels and lock dimensions are governed by a number of factors, including types and volume of probable future tonnage, types and sizes of vessels and tows in general use on connecting waterways, and developments on other waterways that may indicate the type and size of equipment likely to use the new waterway during its project life. It is important that channel dimensions be adequate to handle the traffic projected to use the waterway. U.S. Army, Corps of Engineers guidance (1980) for channel dimensions states that:

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.

Hydraulic conditions at sites tentatively identified for lock construction should be thoroughly investigated in a general river model of the reach with the lock and dam structure in place.

5.1 Minimum Dependable Depth

Dependable project depth is the minimum depth to be provided for traffic expected to use the waterway; it is not the submergence of the vessel or tow. Thus, a" 9-ft channel" provides a dependable minimum depth of water of 9 feet. The majority of inland waterways in the United States have authorized 9-ft channel depths, and because 9 feet is available, except during drought periods, tows are loaded to 9 feet. The locks are designed to accommodate vessels of 9-ft draft.

Minimum depth in a canalized waterway is usually referenced to normal pool elevation, and pool levels should provide project depth and width over all obstructions in the river bed and over the lower lock sill of the next dam upstream. However, in long, narrow navigation pools, where even low discharges cause an appreciable water surface slope, the water surface profile at minimum discharge may be used as the reference plane rather than the pool elevation.

In a pool with only a short length of channel affected by an obstruction, excavation and maintenance of project depth through that reach can result in reducing the required pool elevation. The costs of such excavation should be evaluated in comparison with savings that could be realized in the cost of lands, damages, and construction of a lower dam.

Navigation pool levels should be set to provide a fixed pool elevation with as little variation as possible because stable pool levels enhance reliability of the waterway and simplify development of port facilities. Greater pool stability can be provided with higher dams because high pools are less frequently affected by flood stages.

5.2 Adequate Channel Width

Adequate width for safe, efficient navigation depends on:

a. Channel alignment.
b. Size of vessel or tow.
c. Whether one-way or two-way traffic is to be provided.

If traffic is projected to be light, provision for one-way traffic may be adequate where reaches are relatively straight with good visibility and if passing lanes are provided. A channel for two-way traffic is much safer and permits traffic to move at higher speeds except when meeting or passing.

Minimum channel clearances for one- and two-way traffic in straight reaches are shown in Figure 5.1. In congested reaches with heavy traffic, greater clearances should be provided. The U.S. Army, Corps of Engineers (1980) suggests the minimum channel widths presented in Table 1 be used in straight reaches, with additional width provided in bends. Mathematical ship simulation models are frequently used to evaluate the ease or difficulty of navigating through specific reaches under various channel widths.

A wider channel is required in bends than in straight reaches because vessels and tows take an oblique position with respect to the tangent of the radius of curvature (measured through the center of the tow) in transiting a bend, Figure 5.2. This angle α, termed the drift angle (or deflection angle), varies with:

a. Radius of curvature of the channel.
b. Speed, power, and design of the craft.
c. Wind forces.
d. Whether the tow is empty or loaded.
e. The flow pattern.

The drift angle for downbound tows is larger than for upbound tows, and design of a channel for one-way traffic is, therefore, based on the channel width required in bends for a downbound tow.

Table 1. Recommended channel width

Tow width (feet)	Channel width (feet)	
	Two-way traffic	One-way traffic
105	300	185
70	230	150
50	190	130

5.3 Freedom from Hazardous Currents

Current velocities in the slack-water pools created by navigation dams are lower than in the natural river, and pool elevations are set sufficiently high to provide adequate depth and eliminate hazardous conditions at rapids. However, the locks and dams themselves may create hazards for navigation because:

a. Tows entering and leaving a lock at low velocity have very limited steering power.
b. Spillway releases can cause tows to break up and drift against spillway gates or sink upstream of the spillway.
c. In some cases, hazardous vortices or turbulence may occur in the upper or lower lock approaches due to operation of the filling or emptying systems.

Some restrictions may be required on operation of spillway gates near a lock to reduce hazardous currents. Guide walls and guard walls are usually provided for some distance above and below a lock to permit tows to move along the walls in safety and line up with the lock.

Maximum velocities and maximum channel depth usually occur along the outer (concave) bank of bends, and a lock aligned with the natural deep-water thalweg of the stream will usually be the least expensive. Lock sites in sharp bends and where the structure would deflect a substantial part of the flow from the deep part of the river should be avoided.

5.4 Minimizing Lock Transit Time

Lockages are time consuming and expensive for both users of a waterway and for operators of the locks, and every effort should be made to minimize lockage time in a navigation system. The time required for tows to pass through a lock for lockages in alternate directions (bound upstream, bound downstream, bound upstream, etc.) includes:

a. The time a tow is operating at reduced speed in approaching, entering, and leaving a lock.
b. The time required to break up and reassemble tows made up of too many barges to pass through the lock in one lockage.
c. The time required to close the lower (or upper) lock gates.
d. The time required to operate filling (or emptying) valves.
e. The time required to fill (or empty) the lock chamber.
f. The time required to operate upper (or lower) lock gates.
g. The time required for tow to exit the lock chamber and reach a point where the tow bound in the opposite direction can enter the lock.

Low-lift locks are simpler to design and construct than high-lift locks, but more are needed in a given river reach, and traffic delays are greater. Lockage time is a part of total "trip time," and savings in trip time increases capacity of the waterway. Such savings can be evaluated in monetary terms in the economic analysis. Lockage time can be minimized by:

a. Providing comparatively straight approaches to locks, free from hazardous currents and with adequate sight distance for safe steering.

b. Designing lock filling and emptying systems so as to minimize valve operating time.

c. Providing lock chambers of suitable size for traffic using the waterway to avoid the need for double lockage of a single tow.

d. Minimizing the number of locks in the system.

Miter gates can be opened or closed in about one minute; sector gates are operated more slowly if there is filling or emptying around the gate.

5.5 Terminal Facilities

The location of future terminal facilities should be given careful consideration in planning the location of locks and dams for a new navigable waterway. The deep, wide pool immediately above a dam is favorable for development of harbor facilities; however, the pattern of local traffic should be evaluated. Locating a terminal near a lock, either upstream or downstream, may require a large number of lockages for local traffic that will interfere with through traffic.

Factors to be considered in locating new terminals along a canalized waterway include:

a. Will the pool level be relatively stable?
b. Is there existing industrial development that could be served by the waterway?
c. Are there suitable areas nearby for industrial expansion and terminal development?
d. Are there connecting modes of transportation (railroads, highways)?

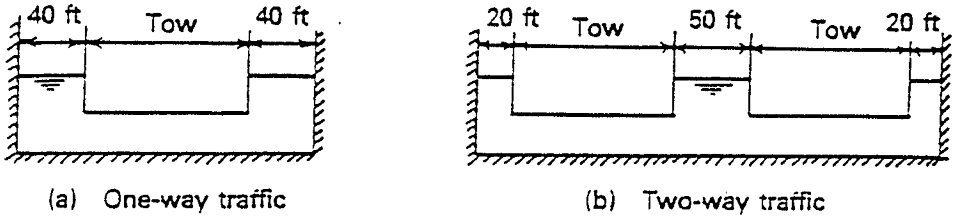


Figure 5.1 Minimum channel clearance in straight reaches. (U.S. Army, Corps of Engineers)

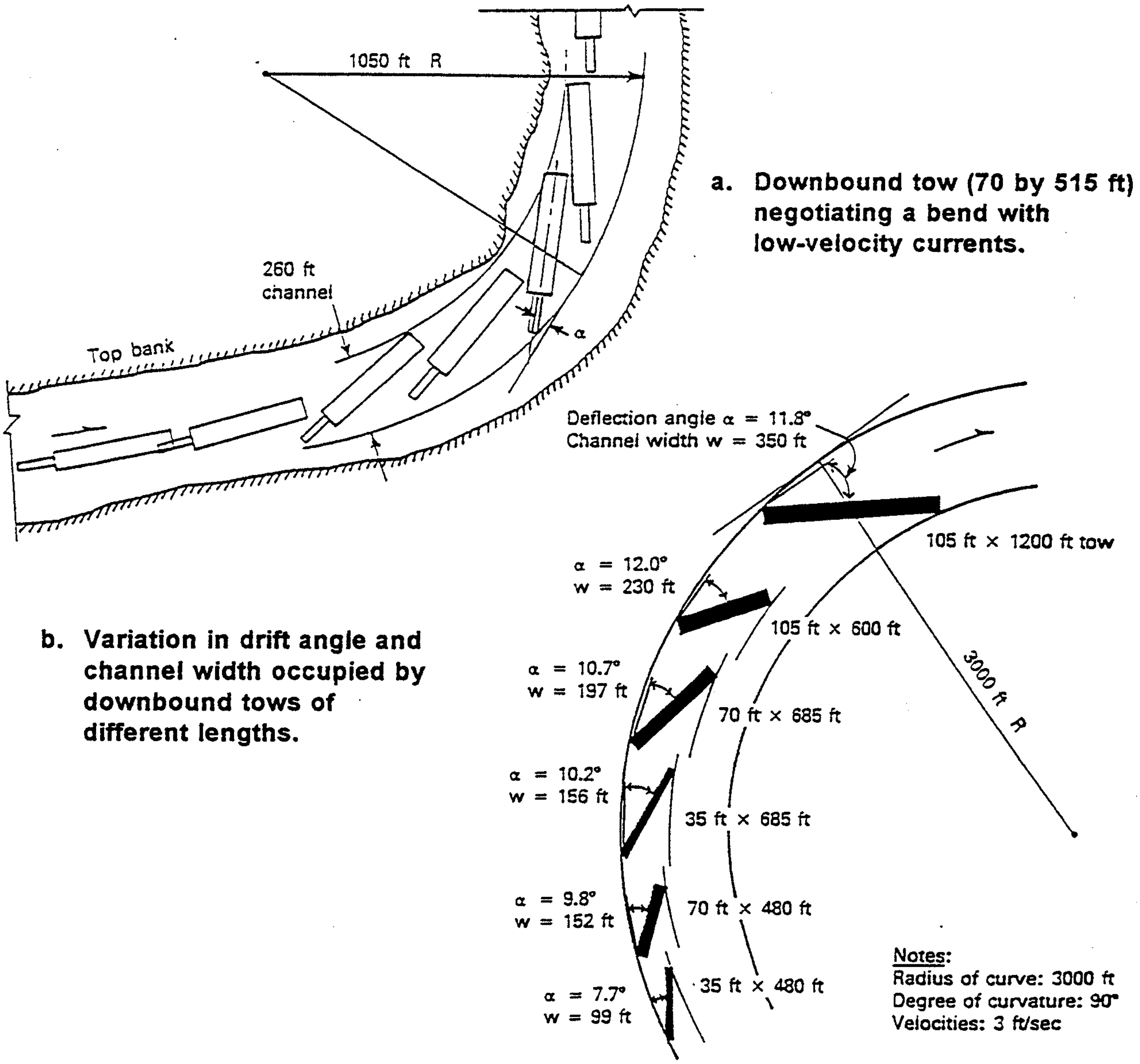


Figure 5.2 Widths required in Bends. (U.S. Army, Corps of Engineers, 1980)

6. OTHER DESIGN REQUIREMENTS

6.1 Flood Stages

Low-lift navigation dams are usually designed to the minimum height required to provide project depth over obstructive reaches of the river:

a. At low discharges, the normal pool level is almost horizontal and at an elevation equal to or somewhat above the low-water stage at the head of the pool.

b. At higher discharges, if the pool elevation remains fixed at normal pool level at the dam, velocities, stages, and water-surface slope at the head of the pool will rise and more land will be flooded, Figure 6.1.

c. The additional depth at higher discharges is not required for navigation, and damage due to flooding adjacent lands may be minimized by drawing down (lowering) the pool level at the dam to where the water-surface profile through the pool provides only project depth over controlling obstructions. Such operation is termed a "hinged-pool" operation and is discussed further in Appendix B.3. The amount of permissible drawdown at the dam is determined by the water-surface slope that would produce limiting velocities for navigation in the lower portion of the pool.

6.2 Drainage

The water surface elevation throughout a navigation pool is permanently above natural low-water stage, Figure 6.1, and for relatively high dams, stages at the dam may be permanently above the highest natural flood level. At the head of the pool, stage will fluctuate between normal pool level and flood stage in generally the same manner as under preproject conditions. This increase in stage throughout a pool may:

a. Interfere with the discharge of sewers, culverts, and tributary streams that formerly discharged freely at low river stages.

b. Result in deposition of silts or sludge in the pool that may block sewers or intakes and raise the bed of tributary streams.

c. Cut off natural drainage paths, requiring rerouting drainage systems or pumping for local runoff to enter the waterway.

d. Raise ground water levels, requiring additional agricultural drainage.

Many drainage problems can be minimized or eliminated by selecting dam sites downstream from major drainage outlets and tributaries.

6.3 Water Supply Intakes

Navigation pools provide reliable depth at water supply intakes, and water quality is an important consideration. If a navigation pool is the source of water supply, sewer outlets should be located downstream of the dam. Special measures may be required to ensure that sediment deposition will not block intakes.

6.4 Sewage Contamination

Aeration provided by turbulent flow through spillways aids in maintaining the dissolved oxygen levels required to support fish life and for aerobic decomposition of sewage. However, immediately above a dam, where pools are relatively deep and velocities are low, wastes may settle out resulting in anaerobic conditions.. Accordingly, navigation structures should not be located downstream of major sewage discharge points. Where structures are located in an urban area, consideration should be given to providing interceptor sewers discharging below the dam.

6.5 Vector Control

In some latitudes, stable navigation pool levels provide an ideal environment for mosquito breeding, particularly if floating debris, dead brush, or aquatic vegetation accumulates in shallow marginal areas. Where malaria is endemic, consideration should be given to fluctuating the pool level about one foot each week in the mosquito-breeding period to strand mosquito eggs, larvae, and pupae along the pool margin. A typical example of such an operation is shown in Figure 6.2 for the Wilson Project of the Tennessee Valley Authority. At Wilson, the pool level is drawn down 1.5 ft below normal pool elevation and refilled each week during the May-September mosquito-breeding season.

6.6 Fish and Wildlife

Impoundment of navigation pools may inundate spawning areas, nesting grounds, and habitat, and dams may block the movement of migratory fish. In designing navigation projects, consideration should be given to recommendations of fish and wildlife specialists as to the effects that various pool levels, dam locations, and operating procedures would have on fish and wildlife resources.

If dams block migratory fish movement, mitigation measures, such as the following, may be needed:

a. Fish ladders for fish to pass around dams.
b. Fish hatchery.
c. Management of fish spawning gravels.

Other mitigation and enhancement measures include:

a. Selective withdrawal of water from various depths in the pool to control temperatures of downstream releases from high dams.
b. Reaeration measures to meet or improve dissolved oxygen levels required for fish.
c. Modified spillway release patterns to meet fishery requirements.

Stable pool levels can reduce the stranding of fish during low water periods in rivers of highly varying discharge and can benefit wildlife having nests and dens near the shoreline.

6.7 Recreation

Impoundment of navigation pools often improves the recreational potential of a river and creates new opportunities for recreation development, particularly where projects are located in or near urban areas. Consideration should be given to including recreation areas and facilities in navigation projects. However, it should be noted that there can be conflicts between recreational and commercial boating on a waterway. Commercial tows have slow maneuvering and stopping capabilities and can be a hazard to recreationists.

6.8 Hydropower

The feasibility of hydropower development should be considered at all navigation dams. Leakage through the locks and dam, evaporation and other water losses in the pool, and water required for lockages must be subtracted from total streamflow to determine the water available for power production. Except in the case of high-lift structures, the most suitable type of power installation is a "run-of-river" plant that utilizes natural streamflow with essentially no modification by storage.

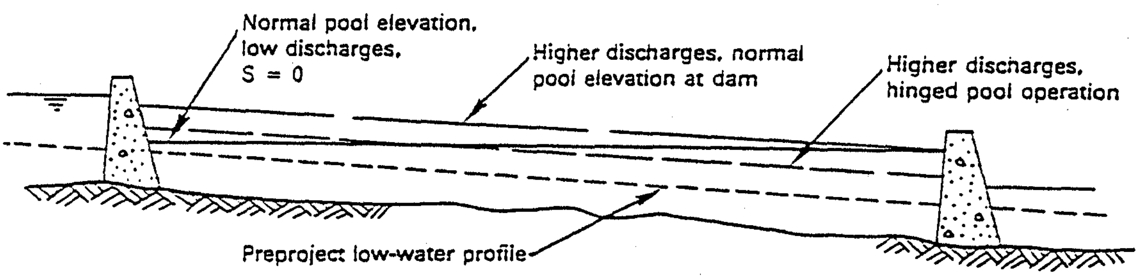


Figure 6.1 Effect of pool operation on water surface elevations.

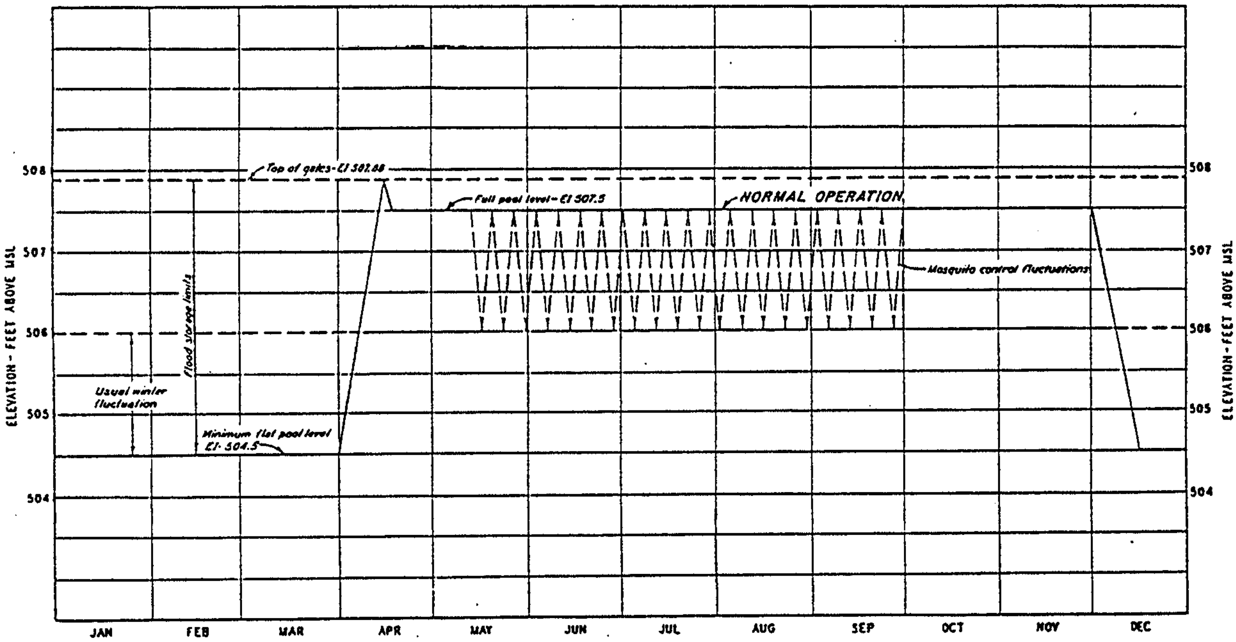


Figure 6.2. Multiple-Purpose Reservoir Operation for Vector Control, Wilson Project, Tennessee Valley Authority. (TVA Monograph 55, Engineering Data, TVA Water Control Projects, 1980).

7. NAVIGATION DAMS

A navigation dam impounds water in a pool to provide navigable depth to the next dam upstream. The spillway or outlet works of such dams is designed to pass flood flows and is regulated by gates to control outflows so as to maintain the pool elevation at an essentially constant elevation except during flood periods. In addition to gated spillway bays, some navigation dams include an uncontrolled concrete weir crest, as at some dams on the Red River, or low overflow embankments in the overbank, as at some dams on the Arkansas River. Navigation dams are of two general types: movable and fixed.

A navigable movable dam is a structure consisting of a number of wickets that can be raised individually to impound a pool at low flows (when traffic uses a lock to pass the dam) and lowered to the streambed to pass flood flows. During high-water periods, traffic can bypass the lock and pass over the dam in the lowered position. Designs of wickets vary, but the structural members supporting the damming surface are hinged to lie flat on a concrete sill at bed level when the dam is open, Figures 7.1 and 7.2. Navigable movable dams are suitable only in special cases where the lift is relatively low, the bed is stable, and there are distinct non-flood and flood periods with river stages high enough for open-river navigation for a significant part of the year.

A fixed navigation dam is a structure with streamflow passing over the top of the dam, through a spillway (either gated or ungated), or through tunnels. Fixed low-head navigation dams are of various types, ranging from the rock-filled timber cribs used in older projects to the low gated concrete crests set at about bed level generally used today, Figure 7.3. The typical design for the Arkansas River navigation project, shown in Figure 7.3, has piers on a broad-crested weir, movable spillway gates, a stilling basin, and protective stone blankets upstream and downstream of the dam to protect the river bed against scour. A similar spillway design was used for dams on the recently completed Red River navigation project, Louisiana. Design of the dam foundation depends on the nature of foundation materials at the site. Design of the piers and operating bridge depends on the elevation of high water and the size and type of gate and operating machinery used.

7.1 Navigable Movable Dams

Navigable movable dams include a navigable pass for passage of tows without locking. A navigable pass must provide sufficient clearance width for the safe passage of traffic and must have sufficient depth for tows of design draft, including depth to allow for overdraft and tow squat. Model studies indicate that a navigable pass should have a minimum cross-sectional area 2.5 times the area blocked by a loaded tow. Current direction should be aligned normal to the axis of the pass, and velocity through the pass must be low enough to permit passage of an upbound loaded tow of the horsepower operating on the waterway. Navigable pass widths at Corps of Engineer projects range from 200 ft on the Ouachita River to about 1200 ft on the Ohio River.

The Corps of Engineers still operates a few dams with older wicket gate designs, such as shown in Figure 7.1, on the Ohio and Ouachita Rivers and the Illinois Waterway, but such designs are no longer being constructed.

Canalization of the Ohio River was initially completed in 1929 with 50 low-lift locks and dams, all with wooden wickets and a 110- by 600-ft lock chamber. Replacement of those structures with 19 locks and dams was initiated in 1954. Eighteen of the replacement structures are high-lift fixed dams with 1200-ft locks. The last, and most downstream, structure is Olmsted Locks and Dam currently under construction about 16 miles above the confluence of the Ohio and Mississippi Rivers, replacing the old Ohio River Locks and Dams 52 and 53. Olmsted is the only replacement dam on the Ohio River that uses wickets, Figure 7.4. A unique centrally-controlled hydraulic lifting mechanism was considered to raise the 220 wickets for the Olmsted project, Figure 7.2. However, a manual operating system from boats is planned at this time. The Olmsted wickets will be 25.5 ft high and 9.2 ft wide; wooden wickets at the existing dams are about 14 ft high and 4 ft wide. At Olmsted, the wickets will be placed on a concrete sill with a baffled stilling basin with a sloping endsill, Figure 7.2.

7.2 Spillways

Spillways for low-lift navigation dams are usually designed with sufficient flow capacity to limit the backwater effect of the structure to about one foot for the project design flow. Where raising flood levels more than one ft is locally acceptable, as for some dams on the Red River, it may be more economical to obtain additional flowage easements and use fewer spillway gates, as discussed in Appendix B. Such spillways for low dams are usually of the broad-crested type because flow over the spillway is influenced by tailwater levels for most operating conditions.

Spillways normally are set near the river bed to maximize capacity and reduce backwater and extend across the entire river. The gate sill and stilling basin may either be level across the channel or set at different elevations across the stream to conform to the natural river cross section, preserve natural flow distribution across the channel, and minimize obstruction of the flow area when the gates are fully open, Figure 4.1. The spillway at Lock and Dam 4 on the Arkansas River was set at two elevations, with the high section at the opposite bank from the lock (where deposition occurred prior to project construction). After 15 years of operation, the benefits of the stepped crest are considered negligible, and a level crest elevation would be recommended (Corps of Engineers, 1987).

Spillways for navigation dams sometimes include uncontrolled overflow crests, depending on local conditions and optimization studies analyzing the costs of providing additional spillway gates needed to pass the design flow with about one foot of swellhead at the structure vs the combined costs of fewer gates and flowage easements needed due inundation of additional lands upstream. Also, it is sometimes desirable to provide additional flow capacity on the overbank to minimize backwater effects. Overflow embankments on the overbank are set as close to the overbank ground level as feasible to best utilize flow capacity of the overbank, and such embankments should be at least three ft above the navigation pool to allow for variation in pool levels, wind setup and wave runup.

On rivers where low dissolved oxygen levels during low-flows present a water quality problem, special measures may be needed to reoxygenate water discharged over the spillway. At Locks and Dams 4 and 5 on the Red River, Louisiana, one spillway bay has a hinged crest which draws warm water from the surface of the pool and discharges it onto a baffled chute, as discussed in Appendix B.4. Turbulence on the baffled chute increases dissolved oxygen levels.

Hydraulic models of spillways are employed to determine:

a. Minimum crest length in the direction of flow and shape of the downstream face of the sill to ensure that there is no separation of the nappe from the sill and no undulating jet action for all partial gate openings for the expected range of pool levels and various stilling basin elevations, and no serious negative pressures on the gate sill.
b. Optimum shape of gate pier nose.
c. Spillway rating curves.
d. Stilling basin performance curves for the expected range of tailwater levels.
e. Riprap requirements downstream of the stilling basin.

Low-head navigation structures have four possible flow regimes, as shown in Figure 7.5, depending on the effects of gates, tailwater elevation, and flow through the structure.

7.3 Spillway Gates

Various types of spillway gates are used, depending on spillway operating requirements and costs. If more than one type is suitable for a particular case, selection is based on cost. Where passage of ice or debris downstream through the dam requires wide gate openings, submergible gates (roller, tainter, or vertical lift gates) are used. These gates can be raised for normal operation, with discharge under the gates, but can be submerged below upper pool level to pass ice or debris over the top. If the range of stage is large, vertical lift gates may be more economical than tainter gates with very long arms. Where passage of ice or debris is not a problem, either tainter gates or vertical lift gates are generally used. Hinged crest gates and baffles on the downstream spillway face were used at two dams on the Red River Waterway where low dissolved oxygen levels were a problem in extreme low-flow periods. The hinged crest gates draw water from the warm surface level of the pool and discharge it onto the baffled spillway face where turbulence oxygenates the flow.

Tainter gates are a segment of a cylinder mounted on radial arms that rotate on trunnions embedded in piers on the spillway crest, Figure 7.6a. The gate consists of a skinplate over a system of beams that transmits the water load on the gate to the radial supporting arms. The gates may seal against the top of the sill, or may lower past the sill for passage of water (and ice and debris) over the top of the gate, Figure 7.6b. Gates designed for submergence have the skinplate extended over a rounded crest and down the lower face of the gate. Tainter gates are raised and lowered by chains or cables at the ends of the gates and are less resistant to torsion than are roller gates, but for short spans they are less costly than roller gates of comparable height. It is essential that these gates be designed to be raised above the design flood flow line so as not to raise flood levels and not to endanger the gate. Clearance is usually from one to 5 ft above the probable maximum flood. It is desirable, but not mandatory, that the trunnions be above high water, and trunnion elevation is set above most flood levels, so that submergence occurs only 5 to 10 percent of the time. Gate vibration has been a problem when tainter gates operate under submerged flow conditions at some dams. Tainter gates have been designed with heights in the range of 75 ft and lengths of up to 110 ft. Where extremely long arms would be required, it is not practicable to use tainter gates.

At Marseilles Lock and Dam on the Illinois River, non-submersible tainter gates on the spillway were replaced by submersible gates in 1987 to skim ice and debris over the top of the gates with much smaller discharge than required to draw the material under non-submersible gates. The gates, Figure 7.7a, were model tested with two spillway profiles, Figure 7.7b, and test data indicated that the crest shape had little or no effect on discharge characteristics of the structure. The data indicated that the Type 1 crest would be unstable due to vibration.. The Type 2 crest was adopted, and the gate was modified to extend the gate end shields near the piers, Figure 7.7c, to decrease the clearance between the shield and pier from 4 inches to 0.5 inches; the gate to sill clearance of 1 inch was maintained. The gates have operated for several years without vibration problems.

Roller gates are metal cylinders with ring gears at each end that travel on inclined metal racks on the piers, Figure 7.8. The roller gate is braced internally and acts as a beam to transmit the water load to the piers. Water, ice, and debris can be passed over the gate, and the gate can be raised to pass water under the gate. Roller gates are raised and lowered by a chain around one end of the gate operated by a hoist mounted in the pier. Water can be admitted to or released from the interior of the gate to change the gate's buoyancy, and the rolling movement of the gate and limited friction contact at the seal make roller gates easy to operate. They have been designed with heights up to 30 ft and lengths up to 124 ft on pile foundations and 150 ft on rock foundations.

Vertical lift gates have a skinplate over horizontal girders that transmit the water load to the piers, Figure 7.9. High piers are required for the gates in the fully-raised position above high water level. To minimize gate vibration, the gate lip in contact with the flowing water is kept as narrow as possible. Vertical lift gates are mounted on wheels or rollers to permit movement under water load, and are raised by chains at both ends, with the entire weight carried by the chains. The gates move vertically in slots in the spillway piers and seat on steel sills mounted flush on the spillway crest. Vertical lift gates have been designed for heights up to 60 ft and for spans in excess of 100 ft. When very high gates are required, a vertical lift gate may be designed in two or more horizontal sections (leaves) to reduce the required hoist capacity, reduce pier height, reduce damage to fingerlings passing downstream, facilitate passing of ice and debris, or simplify design of the ogee crest.

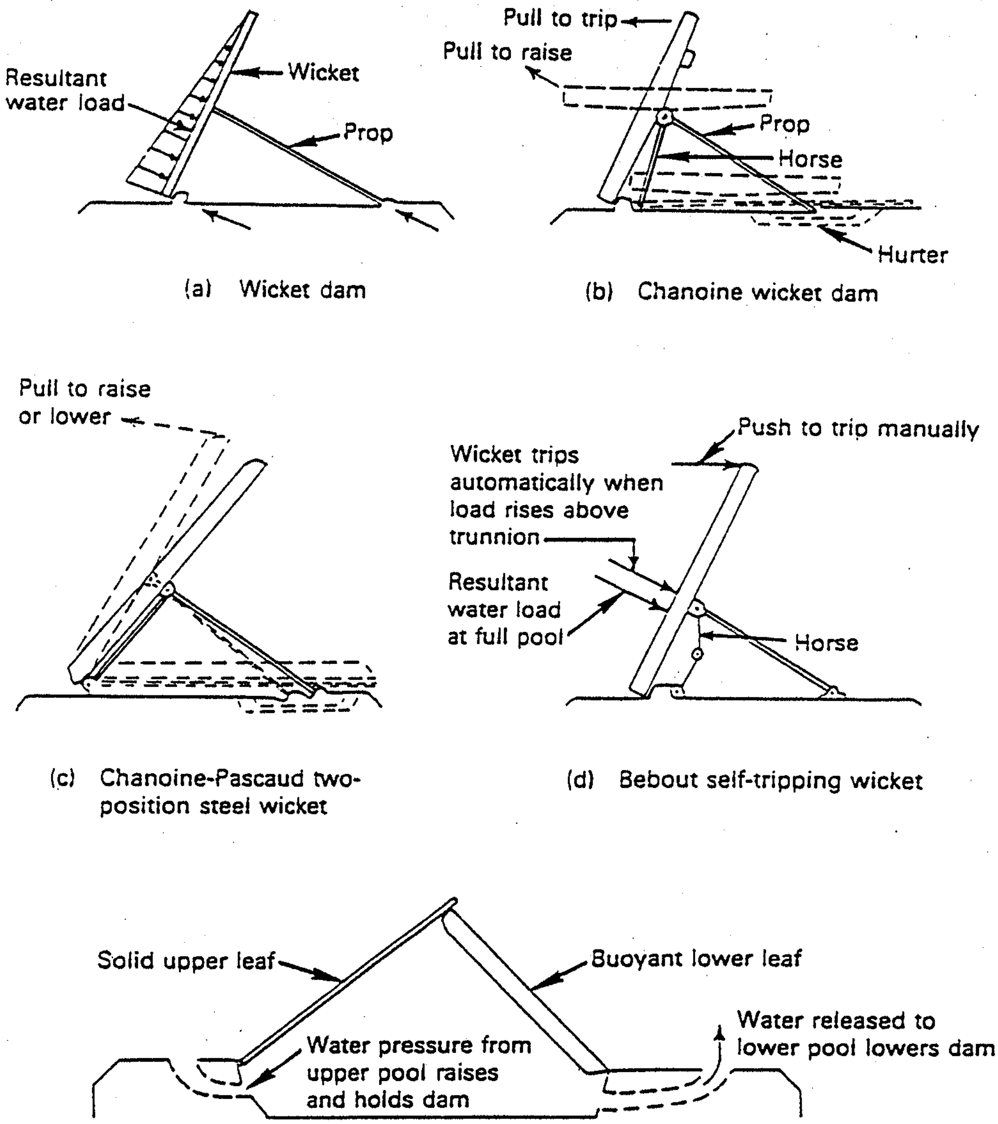
Hinged crest gates, or flap gates, of the type used on some recently constructed spillways on the Red River waterway, can be used to pass warm water from the upper level of the pool or to pass debris and ice. Hinged gates consist of a skinplate that transmits water pressure to an internal system of girders. They are operated by a hydraulic piston and rotate about a hinge on the weir crest and form a part of the crest when in a lowered position, Figure 7.10. The hinged crest gate used at dams on the Red River waterway is described in Appendix B.4.

7.4 Spillway Piers

The nose of ogival spillway piers on the Arkansas River project were shaped so that pier radii meet to form a 90-degree angle at the leading edge of the pier, Figure 7.3c, and a structural steel angle was embedded into the nose to protect the piers from damage when hit by loose barges. It has been found that the sharp steel angle tends to rip open barges, causing them to sink upstream of the piers. The steel nose edge has proved very efficient hydraulically for uniform gate openings, but when there is a difference in gate settings, it causes a separation of flow from the face of the pier on the side passing the greater discharge. An ogival shape with rounded leading edge is recommended (Schmidgall, 1995).

7.5 Ice

In cold climates, such as on the Upper Mississippi River, traffic ceases for several months during the winter period. However, the locks and navigation dams are operated throughout the winter to pass winter flows and ice. Passing ice is handled in different ways at the various projects. The primary factor controlling ice passage appears to be velocity of the ice as it approaches the structures. To maintain pool levels during periods of low flow, it is preferable to pass ice over the top of the spillway gates or through the lock.



(a) Wicket dam

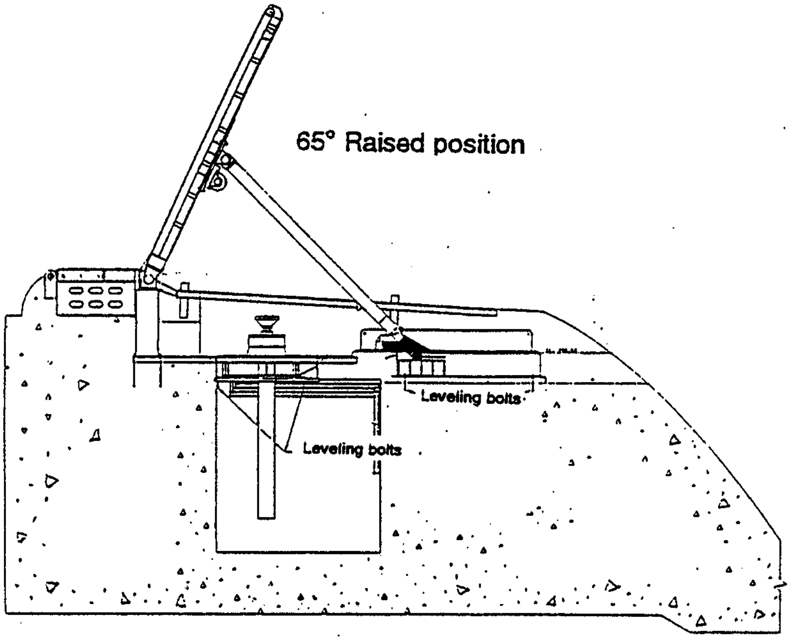
(b) Chanoine wicket dam

(c) Chanoine-Pascaud two-position steel wicket

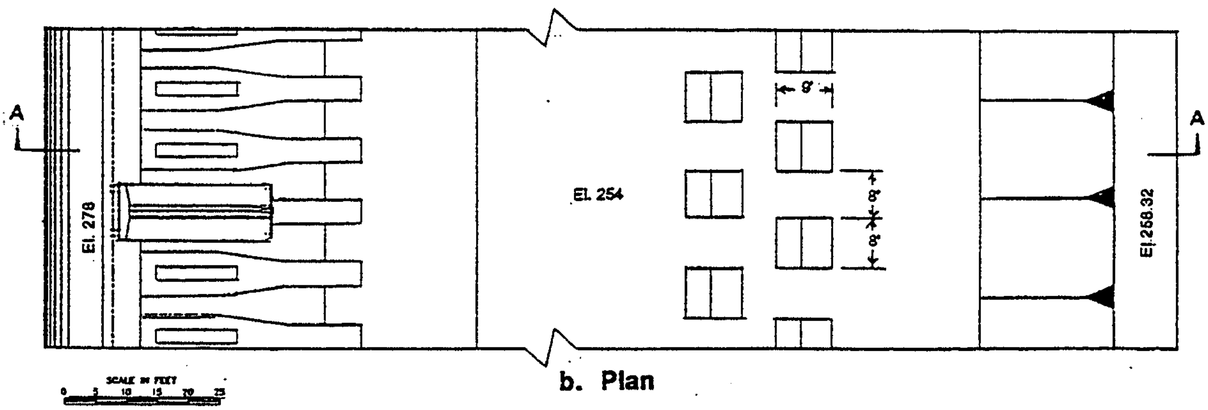
(d) Bebout self-tripping wicket

(e) Bear-trap dam

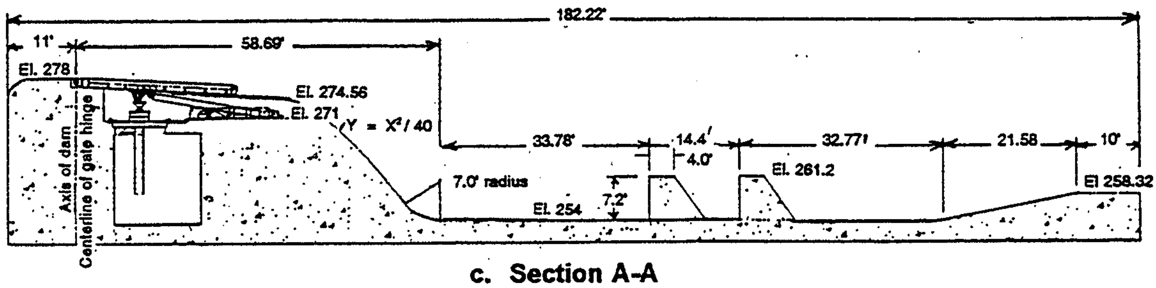
Figure 7.1 Movable dams. (U.S. Army, Corps of Engineers, 1952).



a. Section through wicket gate and weir



b. Plan



c. Section A-A

Figure 7.2 Navigable movable dam (wicket gates), spillway and stilling basin, Olmsted Locks and Dam, Ohio River.

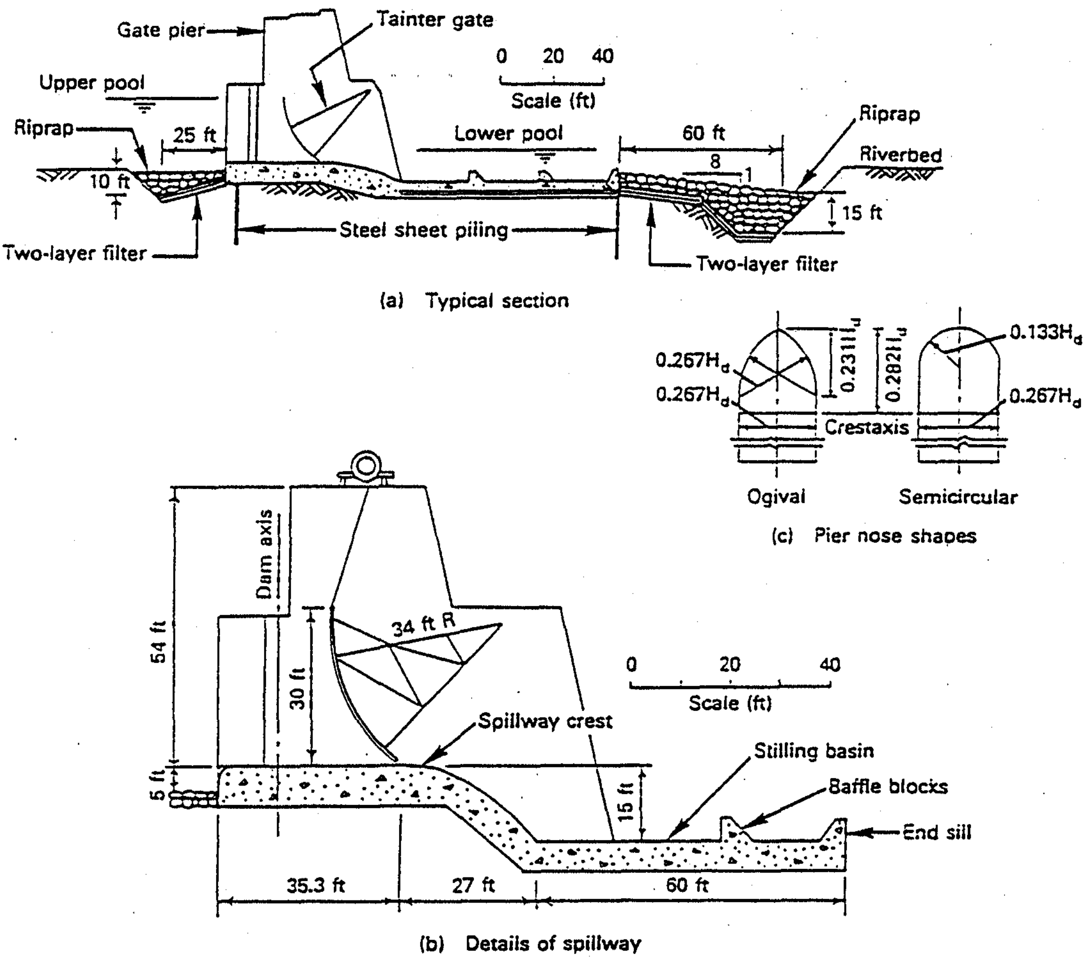


Figure 7.3 Typical non-navigable movable dam (gated spillway), Arkansas River. (Grace, 1965)

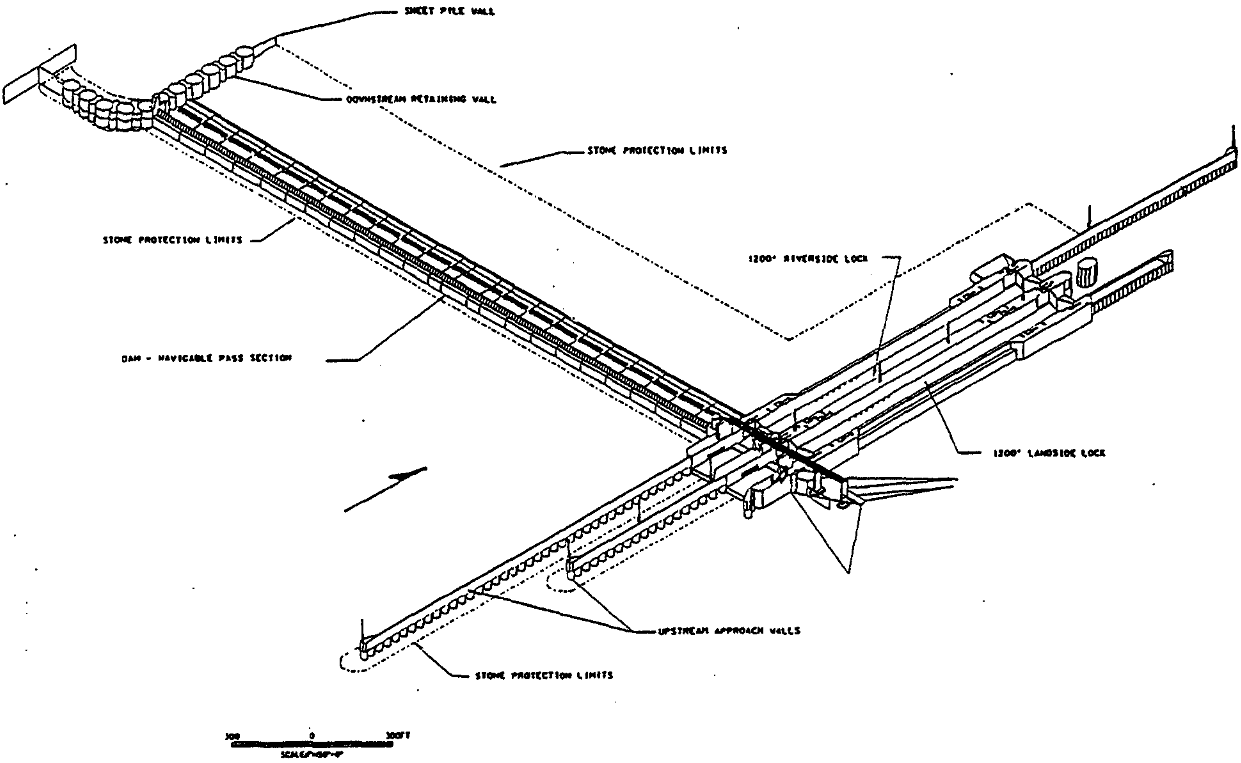


Figure 7.4. Olmsted Locks and Dam, Ohio River.

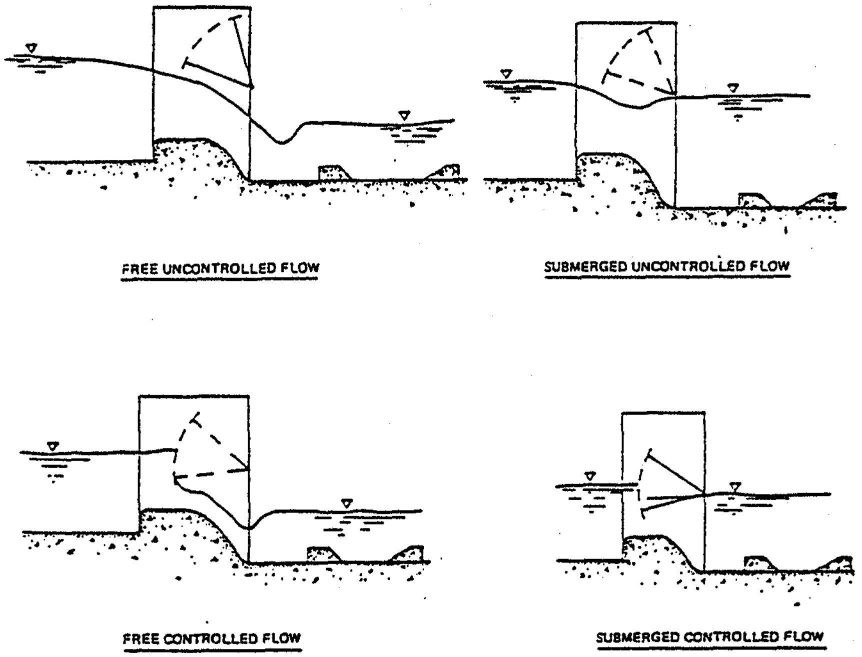
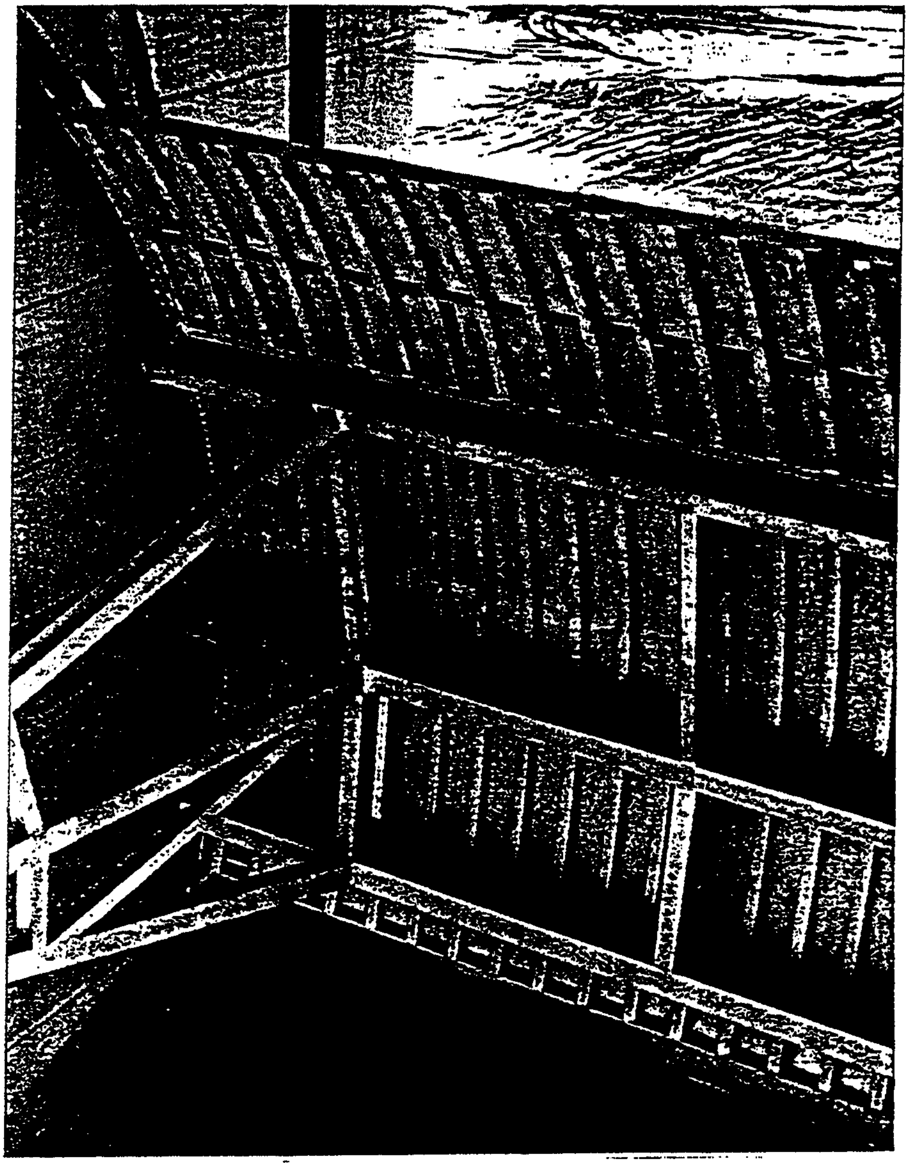
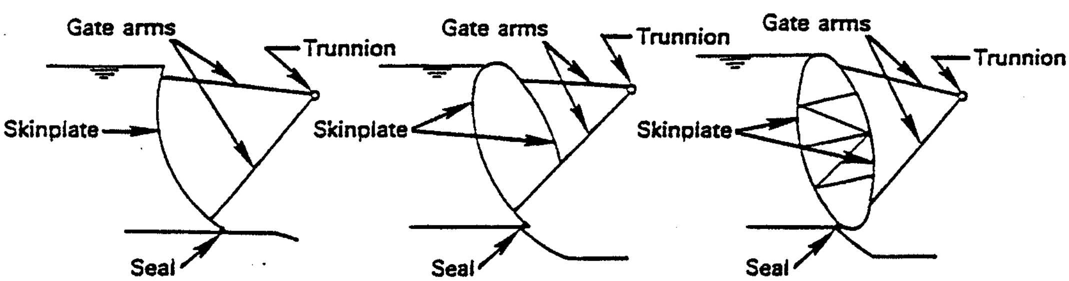


Figure 7.5. Possible flow regimes, low-head navigation weirs. (Corps of Engineers, 1987)

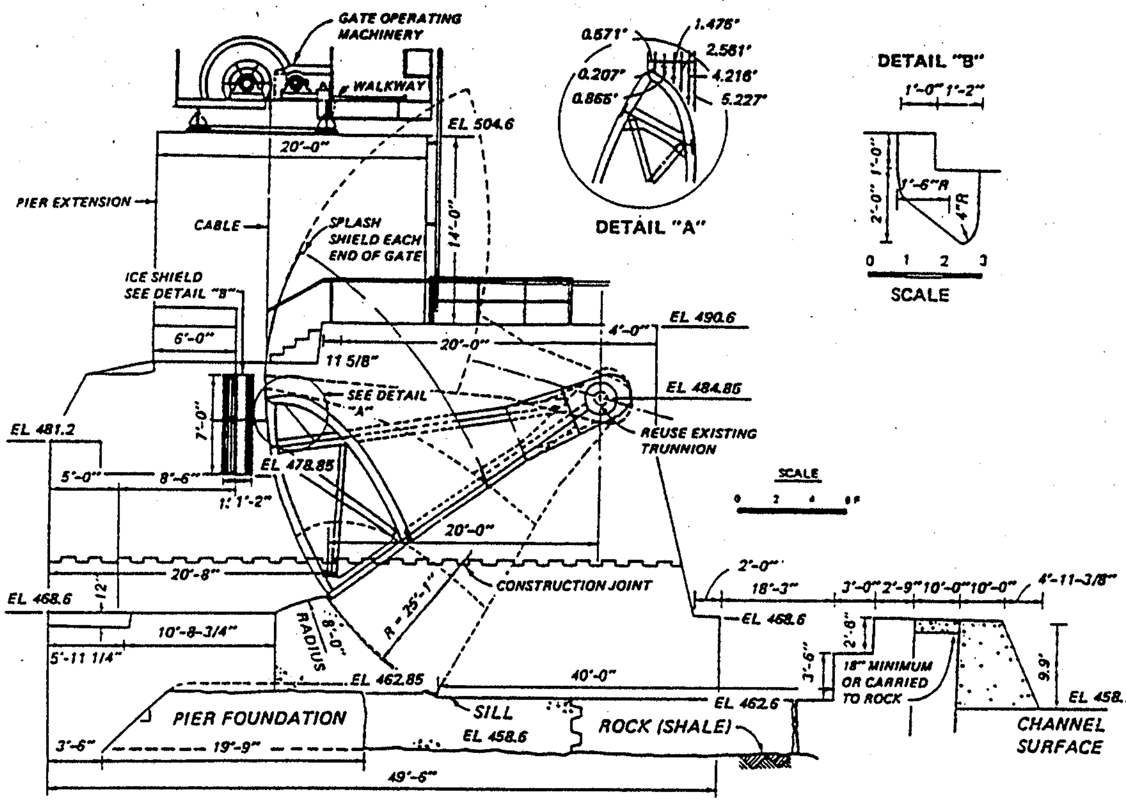


a. Tainter gate on spillway crest (downstream view). Gate is operated by a chain and travels in groove on pier face, Fort Randall Dam, Missouri River.

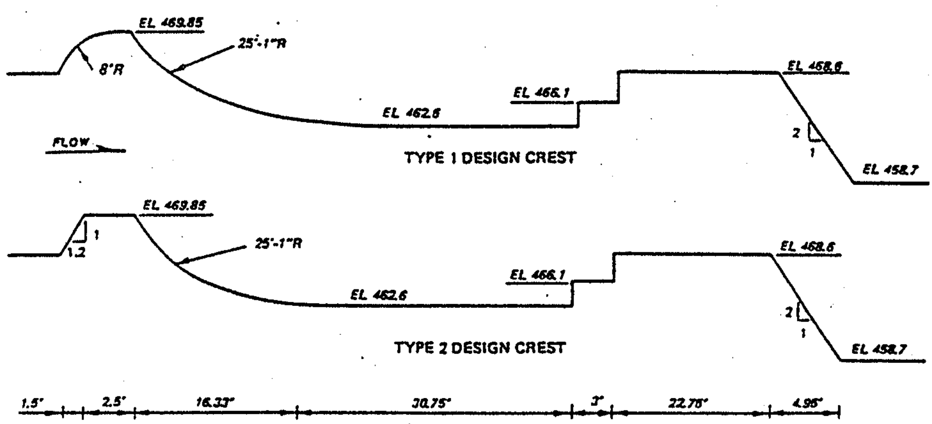


b. Tainter gate seals. (U.S. Army, Corps of Engineers, 1952)

Figure 7.6. Spillway tainter gates.

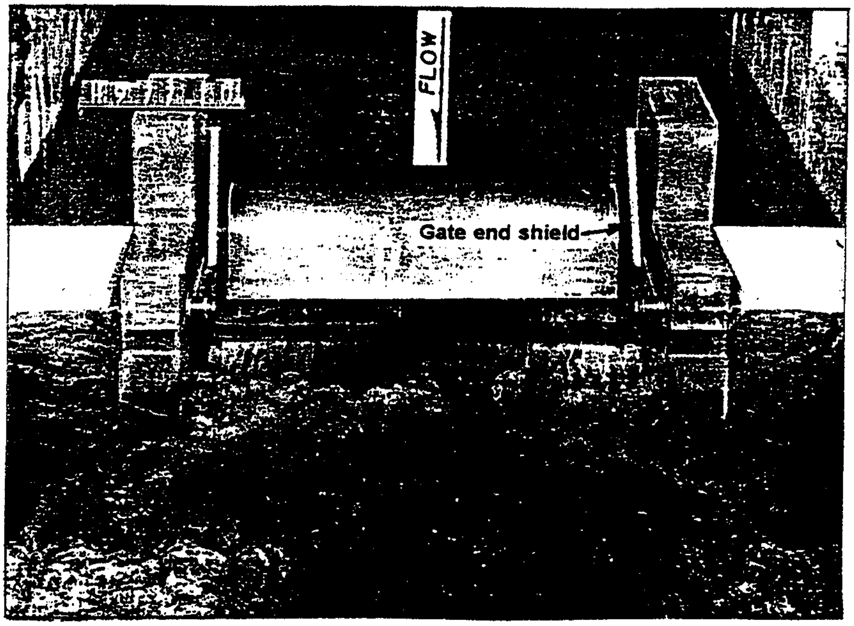


a. Details of submersible tainter gate.



b. Spillway and stilling basin for submersible tainter gate.
Alternative spillway crest shapes tested.

Figure 7.7 Submersible tainter gate, Marseilles Lock and Dam, Illinois River. (Cooper, 1989)



c. Marseilles model; flow under submersible gate. Gate open 7 ft

Figure 7.7 Submersible tainter gate, Marseilles Lock and Dam, Illinois River. (Cooper, 1989)

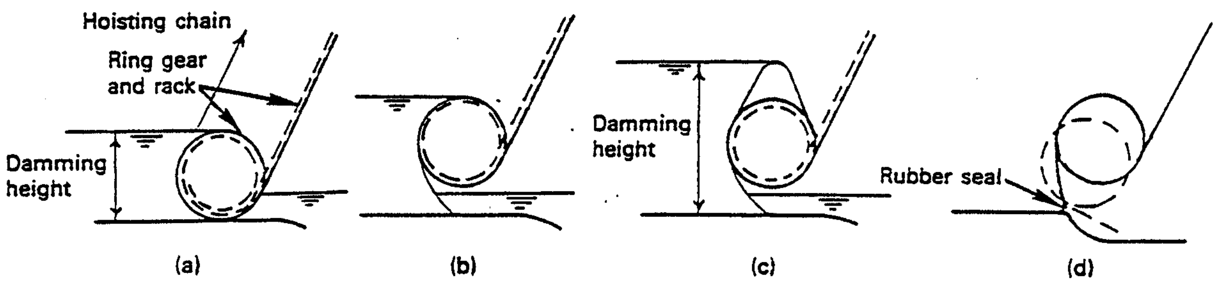


Figure 7.8 Roller gates. (U.S. Army, Corps of Engineers, 1952)

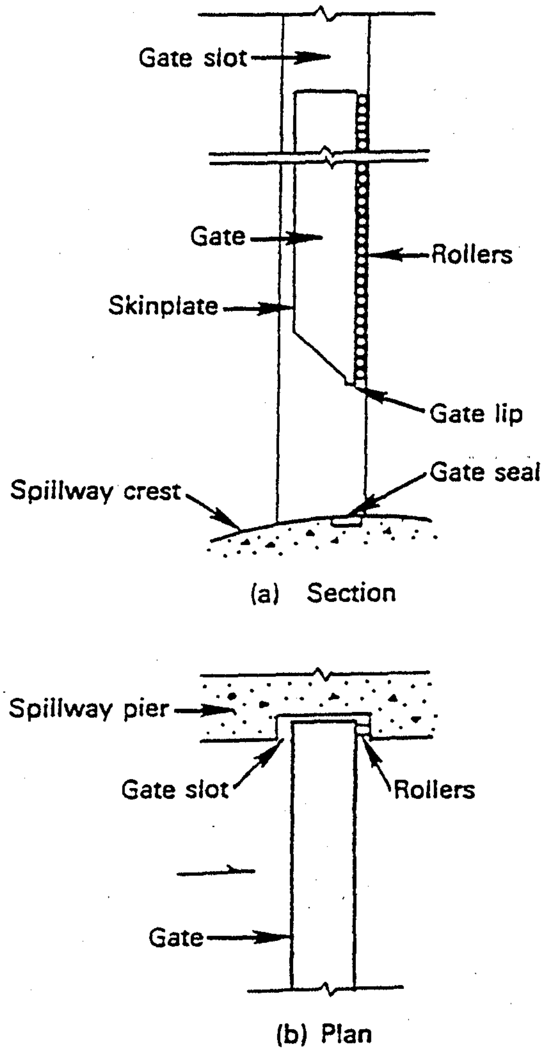


Figure 7.9 Typical vertical lift gate.

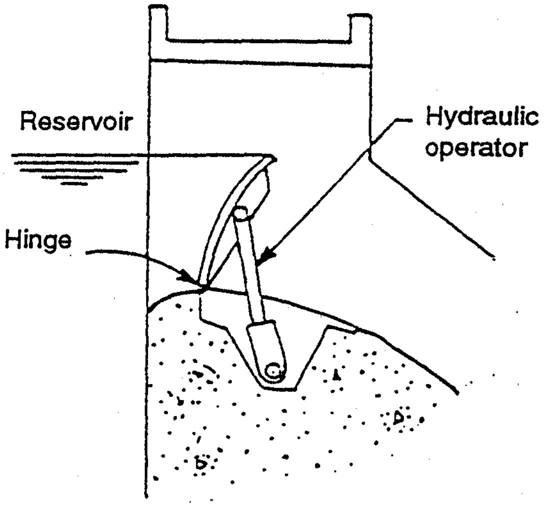


Figure 7.10 Typical hinged (flap) gate.

8. NAVIGATION LOCKS

8.1 General Considerations

Lock Location. In canalized waterways, the navigation lock is usually located near one bank at the end of the dam so that:

a. Spillway length is maximized.
b. Adverse effects of spillway currents on river traffic is minimized.
c. Pilots can approach the lock by traveling along the protected area near the shore.

In canals, the lock often occupies essentially the entire canal width and acts as the dam. Typical layouts for locks are shown in Figure 8.1, and a general layout of a low-lift navigation lock and dam is shown in Figure 8.2.

In general, where two locks are provided (twin locks of equal size or a main lock and smaller auxiliary lock), it was customary to place the locks side by side, with a common center wall, as shown in Figure 8.1. However, at the new Melvin Price Locks on the Mississippi River, replacement for Lock and Dam 26, the two locks (a 1200-ft main lock and 600-ft auxiliary lock) are separated by a 350-ft spillway section with two gate bays, Figure 8.3 The 350 ft separation extends from the inside face of the land lock to the inside face of the river lock and was provided for more efficient use of the lock and higher traffic capacity. The separation distance was based on operation studies and recommendations from towboat pilots for the minimum distance between locks with two tows passing, one tow approaching the locks and a second tow departing.

Pilots must have a clear view of the lock entrances because momentum of a tow when it is slowing down is difficult to control due to inertia and low power. Minimum sight distance in a lock approach of one mile is usually sufficient for safe operation, permitting tows to align with the lock before reducing speed. Model studies are conducted of tow operation in lock approaches to investigate potential operation problems, and the views of rivers pilots are taken into consideration.

The exact location of a lock depends on such factors as:

a. Configuration of the river reach.
b. Shape of the channel cross section.
c. Hydraulic conditions at the site.
d. Bank elevation and stability.
e. Foundation conditions.

Straight reaches of river are more desirable sites for navigation locks and dams than bends because they are easier to navigate. However, straight reaches of alluvial rivers often tend to be unstable, and adequate depth in the downstream lock approach may be hard to maintain.

Adverse cross currents from spillway discharges also may present problems to traffic in the lock approaches. A pair of locks located on the deep side of a bend, Figure 8.4, may block so much streamflow that spillway operation results in undesirable currents in the upper lock approach. The upstream guide wall, as well as the lock itself, can adversely affect flow conditions. Cross currents in crossings may also interfere with tows approaching a lock.

<u>Cofferdams for Construction</u>. Navigation structures are usually constructed in a series of stages so that river flows can be passed during construction and in some cases, such as during construction of replacement for Lock and Dam 26 on the Mississippi, tows can continue to use the river during construction. Consideration must be given to potential problems with cofferdams during the construction period when evaluating alternative sites for locks and dams and also when evaluating alternative schemes for cofferdams at a particular site, including:

a. The number of cofferdam stages and the extent of each stage. Typical cofferdam layouts are shown in Figure 8.5a. Twin 1200-ft locks at the Smithland project on the lower Ohio River (replacement for Locks and Dams 50 and 51) are shown under construction in a cofferdam in Figure 8.5b.

b. Passing navigation traffic through the construction reach while cofferdams are in place if there is commercial navigation on the river prior to project construction.

c. Seepage into the dewatered area inside the cofferdam and related pumping requirements.

d. Frequency of flow at which the cofferdam would be overtopped and flooded.

e. Difficulties associated with passing high flood flows through the construction area while the cofferdams are in place, including estimated scour with different cofferdam configurations.

f. Cost of alternative cofferdams schemes, including the cost of dewatering, cleanup, and repair associated with overtopping of the cofferdam.

The three-stage cofferdam scheme for construction of the replacement locks and dam for Lock and Dam 26 on the Mississippi River is shown in Figure 8.6. Construction began from the west bank of the Mississippi River, with the Stage I cofferdam which enclosed 6.5 spillway bays. The Stage II cofferdam enclosed the 1200-ft lock riverward lock and two half gate bays (one-half gate bay on each side of the lock), and the Stage III cofferdam enclosed 1.5 gate bays and the 600-ft landward lock.

For construction of Dardanelle Lock and Dam on the Arkansas River, 3-stage and a 4-stage cofferdam schemes were considered. The 4-stage plan, Figure 8.7, used larger diameter cells and required less sheet piling than the 3-stage plan. It was cheaper to construct and, therefore, was selected.

Navigation projects can sometimes be constructed off-channel, as was done for locks and dams on the Red River where 36 cutoffs were constructed to realign the channel for navigation. (The 280-mile navigable reach was shortened 50 miles, or 18 percent.) Locks and dams were constructed on the alignment of cutoffs that were part of the overall plan for stabilization and rectification of the future navigable channel. After completion of the locks and dams in the dry, connecting channels were excavated to the river, the old river channel was closed off at the upstream end, and the river was diverted to the new alignment through the cutoff, Figure 8.8.

Access to Construction Site. Ease of access to the project site affects project costs for construction and also for operation and maintenance after project completion. For projects in remote areas, the cost of constructing access roads may be a large part of total project costs. Availability of waterborne and overland transportation systems and power facilities all affect project costs.

Availability of Construction Materials. The availability of construction materials of sufficient quality and in sufficient quantities for project construction within economic distance of the construction site must be investigated in the planning process. When materials such as coarse and fine aggregate and protection stone are not available locally, they must be brought to the site at higher cost.

8.2 Lock Design Criteria

Lock Size and Number of Locks. Consideration of the types of navigation equipment projected to use the canalized waterway, type and volume of projected traffic, and economic studies including project costs and estimated navigation benefits all influence:

a. Lock chamber size.
b. Optimum filling time (whether or not a fast filling and emptying is needed).
c. Whether or not one or two locks are required at each dam.

In some cases, the size of tow that can be physically accommodated at critical channel points along a canalized river may limit the size of tow using a lock and size of lock chamber. Standard usable lock dimensions in the United States are given in Table 2.

Table 2. Usable Lock Dimensions (feet)

Width	Length
84	400
	600
	720
	800
	1200
110	600
	800
	1200

Where a single lock is used, traffic will be interrupted when the lock is closed for maintenance or repair; however, this may not be a major problem if traffic is highly seasonal and maintenance can be scheduled in the off-season. Two locks increase reliability of the system. If one lock is out of service (due to an accident or for maintenance) some traffic can continue to use the one operable lock.

If economic studies do not justify construction of two locks initially, it may be desirable to include some works (such as the upstream lock gates) in initial construction to minimize costs of adding a second lock in the future.

Lock Lift. Lift is one of the first and most important design criteria to be established in planning a canalization project. Maximum lock lift is the vertical distance from the upper pool normal water surface elevation above the lock to the low-water surface elevation below lock; it is the range of water surface levels in the lock chamber Figure 8.9. The lock lift and upper pool elevation must provide adequate and safe depth for navigation over all obstructions throughout the pool and over the lower gate sill of the next lock upstream. The cost for one high-lift lock may be less than the combined cost of two low-lift locks of equal total lift, but the design is usually more complex.

Lift is the major factor governing the type of filling and emptying system used for a particular lock, and locks are generally classified by lift as follows:

Low-lift lock	Less than 30-ft lift
Intermediate-lift lock	30- to 60-ft lift
High-lift lock	More than 60-ft lift

All new high-lift locks in the United States are based on either Lower Granite or Bay Springs manifold systems, Figures 8.19 and 8.39.

8.3 Lock Types

Locks are of various types, and the design used at a particular site is usually determined by foundation conditions and costs. If there are no unusual foundation conditions, gravity locks are usually the most economical type to design, construct, and maintain due to simplicity of design, the relatively small amount of skilled labor required for construction, and low maintenance costs of the thick sections. Reinforced concrete lock wall design is used for walls at gate bays and approach walls and is similar to design of reinforced concrete retaining walls. Gravity walls are reinforced at thin areas, and the dry-dock lock is a reinforced-concrete structure. Approach walls, abutments, the area around culverts, filling and emptying laterals, and other parts of most modern locks are of reinforced concrete.

Gravity mass concrete locks can be designed for soil, rock, or pile foundations and have few structural limitations as to height or lift. Base width of walls must be sufficient to prevent overturning and sliding and overstressing the foundation. Top and intermediate widths of walls must provide a section to withstand the wall stresses and provide space for filling and emptying systems, anchorages for gates, operating equipment, temporary closure structures, and other machinery. Disadvantages of gravity structures include loads that may be heavy with respect to supporting capacity of the foundation materials and the possibility of unequal settlement of adjacent or opposite monoliths that may result in misalignment or damage of movable structures and operating machinery.

One of the newer innovative designs used on the Kanawha River places the filling and emptying culverts in the lock chamber floor and uses roller-compacted concrete for lock chamber walls between the gate monoliths. This is discussed further in Section 11.

Dry-dock type reinforced concrete locks are used where foundation conditions preclude use of a gravity design and where the use of a pile foundation is not practicable. The lock consists of relatively thin lock walls constructed integrally with a thick floor slab, designed to act together as a monolith, each being heavily reinforced to distribute loads. The dry-dock type lock can be unwatered for inspection and repair without fear of a blow-out and loss of foundation material; however; adequate provision must be made to offset the buoyancy effect of the structure.

Steel sheet piling locks are a combination of sheet piling with one or more other types of construction. For temporary locks and waterways that do not warrant costly construction, steel-sheet piling can be used for the walls between gate bays and for the approach walls. The piling is driven in a straight line, and any offsets along the face of the wall can be eliminated by using timber fenders bolted to the piling at levels where the tows usually rub against lock walls. Locks of this type have a relatively short useful life of about 15 to 25 years.

Combination-type locks combine several types of construction in one design. Where a considerable amount of sound rock must be excavated, a layer of reinforced concrete may be constructed adjacent to the vertical face of the rock to form the lower portion of the lock walls. The concrete is anchored to the rock by steel dowels grouted into drilled holes. The upper portion of the walls is of gravity design.

For low-lift projects and in canals, levees may form part of the lock walls between gate bays. Walls of concrete or sheet piling can be constructed to a height to accommodate navigation a large percent of the time, but the gate bays, gates, and levees should be built to above the maximum stage at which lockage is provided. When the walls between the gate bays are overtopped, the levees and gate bays would maintain the pool elevation.

8.4 Lock Depth and Lock Floor

Locks fill and empty through a system of intakes and culverts upstream of the upper lock gate in the lock walls or upper gate sill; culverts in the lock walls; ports in lock chamber walls or on the lock floor; and emptying systems downstream of the lower lock gate.

It is desirable that lock filling time be as short as possible to minimize delay and cost to tow operators. However, there is some turbulence associated with the filling operation, and the lock must be deep enough to provide a "cushion" of water over the filling ports to dampen turbulence so that tows are not damaged and stresses in the hawsers (lines securing tows to the lock walls) are within acceptable limits.

Depth provided in the lock chamber and over the lock gate sills depends on the type and size of vessels and tows using the lock. Lock depth is the usual dimension governing overall lock design, and is usually determined by design requirements for the filling system. The sill elevation may govern the lock floor elevation in some cases because the floor should be at least 2 ft below the sill for operation and maintenance. For a side port system, the required cushion depth over the ports usually controls. For bottom lateral and longitudinal culvert systems, the top elevation of the bottom culverts may control, as they should be no higher than the top sill elevation.

Where foundation materials are erodible, such as sand and gravel, the concrete lock floor is usually subject to downward pressures when water in the lock is at upper pool level and to upward pressures when it is at lower pool elevation or when the lock is unwatered. Accordingly, the lock floor must be designed to withstand uplift due to hydrostatic head or relief wells must be provided. On alluvial streams, a line of steel sheet piling is sometimes driven around the perimeter of a lock under the walls and sills to stabilize the foundation material and prevent movement of material out from under the lock walls.

For locks excavated in rock, a concrete floor may not be necessary if the culverts and ports are located in the lock walls.

8.5 Lock Gates and Sills

Lock gates operate on sills, as shown schematically in Figure 8.9. Miter, roller, sector, tainter, and vertical lift gates are used as lock service gates, and each type has special characteristics that make it the most suitable for any given site. Design of the gate sill varies with the type of lock gate used, and deeper depths over the sill increases locking efficiency.

Miter gates are the most widely used type of lock gate on inland waterways in the United States and are the only gates that cannot be operated (opened and closed) with a differential head upstream and downstream of the gate. Other gate types can be used as both lock service gates and for filling or emptying the lock and can be opened or closed to any position and held at that position. Miter gates and miter gate operating machinery are designed to be under complete control of the gate operator during opening and closing operations, to remain completely closed when in the closed position, and to remain completely open and in the gate recesses when open.

Lock gates are designed for a static hydraulic load and for a temporary hydraulic load which may either add to or decrease the static head and, in extreme conditions, may produce a reverse head. Reverse loads almost always occur as a result of temporary conditions and are of very short duration, except at tidal locks. Most frequently, reverse heads result from temporary lock overfilling or overemptying due to the momentum of water moving in the culverts, and this is generally the most serious temporary loading condition. Loading conditions are as follows:

a. The maximum static hydraulic load on the upstream gate is the load due to difference in water surface elevation of the maximum upper pool and the gate sill elevation.

b. The maximum static head on the downstream gate is the difference in elevation between the maximum upper pool and the minimum lower pool.

c. Temporary hydraulic loads on gates can be caused by wind waves, seiches, surges, waves from propeller wash, ship waves, and tidal action.

d. Temporary head reversal can cause miter gates to be briefly forced open slightly and then slam shut, possibly damaging the gates.

Davis (1989) suggests the following guidance for evaluating temporary hydraulic loads:

a. Use a temporary hydraulic load of 2.5 ft for durations greater than 30 sec for direct or temporary reverse heads no greater than 2.5 ft. This is a minimum value and applies to structural design of all gates, gate leaves, and operating machinery except miter gate operating machinery.

b. Use a temporary hydraulic load of 1.5 ft for durations exceeding 30 sec as the minimum value for design of miter gate operating machinery.

c. Do not use miter gates where a temporary reverse loading significantly greater than 2.5 ft can occur for more than 30 sec.

d. Because overfilling and overemptying can occur on every lock operation, gate operating procedures should be designed to reduce potential reverse heads to nondamaging values, as by starting closure of the filling valves before the lock chamber is full. Automatic controls can be designed so the valves will be about 95 percent closed when the lock chamber is full.

Lock gate sill elevations are set with relation to normal water surface elevation in the adjacent pool, and gate sill elevation controls the draft of tows that can use a lock. For hinged-pool operation, the upper sill must be low enough to provide adequate depth when the pool is hinged. Because of the difference in pool levels and lock lift, the upper gate sill elevation is always higher than the downstream, and the downstream gates are always much higher than the upstream gates, Figure 8.9. For example, Bay Springs Lock on the Tennessee-Tombigbee Waterway has an 84-ft lift, and the upper gate sill is 75 ft above the look floor elevation, Figure 8.10, while the lower gate sill is at the same elevation as the floor.

Greater additional depth is provided over the downstream sill than over the upstream sill because a tow that fills the width of the lock chamber will squat several feet on entering the lock and may strike and damage the sill unless sufficient clearance is provided. Sill elevations are determined by taking into consideration future development of navigation carriers and possible degradation downstream of the lock. To provide greater clearance at the time of construction usually does not increase initial project costs materially, but to provide it later might require temporary closure of the waterway to traffic and costs could be excessive.

As a tow enters a lock, the water displaced by the tow flows out of the lock chamber between the bottom of the tow and the lock sill, and considerable space is required between the bottom of the barges and the sill. When the last water displaced runs out, there is a sudden drop in resistance to the tow's entry into the lock, and if the tow is not at a dead stop, inertia will carry it forward into the upper sill or into the upper lock gates. Towboat captains are aware of this phenomenon and keep their entrance velocities within safe limits. Operation is easiest, safest, and least time consuming with greater sill and lock chamber depths. Safety considerations are worked out with engineering and experienced operating personnel and in consultation with members of the towing industry.

Under-tow clearance for optimum filling time, would be a 23-ft lock chamber depth, or a depth/draft ratio of 2.5. All Corps of Engineers 110-ft locks constructed since 1970 have a sill depth to draft ratio of at least 1.7 (that is, 6 feet of under-tow clearance for a 9-ft channel) or greater. Ideally, depth over the sill of twice the tow design draft (18-ft depth over the sill for a 9-ft channel) should be available 95 percent of the time, and minimum clearance of 1.7 times the draft should be available 100 percent of the time. Most locks have 1 or 2 ft less depth over the sill than in the lock chamber. In cold climates, such as along the Upper Mississippi River, ice accumulates on the bottom of barges; six to eight ft of ice accumulation is not uncommon. The downstream sill should not be more than 3 ft above the chamber floor as there is not much difference between the cost of one foot of sill height and one foot of lock gate height and greater clearance over the sill increases safety.

All gate sills must resist lateral forces, consisting of both earth and hydrostatic pressure, from the bottom of the gates to the sill foundation. Often ports for culvert filling and emptying systems and crossovers for various utilities are located in the gate sills.

<u>Miter Gates</u>. Miter gates consist of two gate leaves, each rotating on a vertical axis in a recess in the face of the lock walls. When open, they are recessed in the lock walls and are flush with the face of the wall, Figure 8.11. When closed, the stainless steel mitered edges of the two leaves meet at the center line of the lock, and the gates are angled slightly upstream with respect to the lock walls so that upstream water pressure contributes to keeping the gates tightly closed and minimizing leakage. The steel gates have a girder framework covered by a skinplate on one or both sides. They are designed with sufficient rigidity so that they do not twist or become warped when rotated through the water.

<u>Tainter Gates</u>. Tainter gate sills are of two types with respect to loading. One, which merely provides a sealing surface for the gate and a top surface to fit spillway characteristics, is used only for narrow lock chambers where the entire gate load is transferred to the lock walls through the end trunnions as at St. Anthony Falls, Figure 8.12. The second type is used for wide lock chambers where end and intermediate trunnion arms transfer their loads to trunnion castings anchored to buttresses attached to the sill. The sills are generally higher than the lock floor area where the gates swing open so that any debris on the floor of the lock will not interfere with gate operation, Figure 8.9.

<u>Sector Gates</u>. Sector gates, shown in Figure 8.13, are used as lock gates where reversal of head occurs for significant periods of time, for example at a location affected by tidal action where the downstream water level is sometimes higher and sometimes lower than the upper pool. Sector gate sills are primarily to form sealing surfaces for the gates when closed and sometimes to provide rolling tracks to carry a portion of the dead weight of the gates.

<u>Lift Gates</u>. Lift gate sills provide a sealing surface and act as a spillway weir.

<u>Emergency Closure Sills</u>. Emergency closure sills provide a sealing surface for such

structures as emergency gates, bulkheads, and so on that are provided to stop flow through the lock chamber if the service gates become inoperative and to close off the lock chamber to permit unwatering for periodic inspection and repair. Emergency closure sills are often outside the intake and discharge ports of the filling and emptying system so that the ports and filling and emptying system can be unwatered for inspection and repair. Bulkhead sills do not resist any part of the bulkhead lateral load, and the sill is designed only to support the weight of the bulkheads and hydrostatic pressures below the bottom bulkhead unit. The bulkhead-type closure provides a positive seal in flowing water without requiring the assistance of a diver at the top of the sill during installation. Emergency closure facilities are discussed further at the end of this section.

8.6 Lock Walls

Lock walls are designated by location and purpose. For a single lock, walls are designated as either land river wall. For two locks side by side, the dividing wall is designated as the intermediate or middle wall. Wall designations by purpose, shown in Figure 8.9, are:

a. Lock chamber walls.
b. Upper gate bay walls.
c. Lower gate bay walls.
d. Culvert intake walls.
e. Culvert discharge walls.
f. Upper and lower approach walls (guide walls and guard walls).

Lock walls always resist part of the gate thrust, and provision must be made to absorb these loads in the walls as well as to provide sufficient space for operating machinery.

The height of lock walls above pool elevation depends on the stage and flow at which navigation ceases, the importance of the waterway, and the value of uninterrupted transportation during high stages as well as on characteristics of the waterway, type of dam, type of lock, balance between initial construction cost and maintenance cost, and other factors. On major waterways, walls are set at sufficient height so that traffic is interrupted only by infrequent flood flows because if published traffic schedules cannot be maintained by shippers during most of the year, or if schedules are subject to numerous interruptions because locks are out of service, projected use of the waterway may never develop.

During the 1993 flood on the Upper Mississippi River, locks were out of service for a total of 77 days (three different closures); seven locks were under water. Costs to repair damage to the navigation locks and dams was estimated at $4 to $5 million dollars. Overall traffic on the Upper Mississippi decreased 30 to 35 percent for 1993, and daily losses to shippers was estimated at $700,000 a day during lock closure.

To protect tows from currents and winds at high river stages, lock walls should be set at least 2 or 3 ft above the stage corresponding to the maximum navigable flow. On the Arkansas River system, it was expected that the river would be navigable for flows up to the 10-yr recurrence interval flood. Velocities for larger floods were expected to be too high for safe and efficient operation of tows. Therefore, the top of lock walls was set at the higher of 10 ft above pool level or 2 ft above the 10-yr recurrence interval flood. Access roads to the locks and dams also were set at the same elevation. The 10-yr recurrence interval flood is also the limit of navigation on the Red River Waterway (140,000-145,000 cfs at Lock and Dam 2 and 120,000 cfs at the head of navigation at Shreveport).

It is usually desirable to set the top of lock walls at as high an elevation as economics of the project permit. Top elevations have been set such that the longest period of traffic interruption during the largest flood of record would not exceed 10 to 15 days. Unless the top of the walls is above flood stage, operating equipment on the walls must be removed each time the walls are likely to be overtopped, and cleanup is necessary after the water has subsided.

Lock Chamber Walls. Lock chamber walls are located between the upper and lower gate bays and enclose the lock chamber. The top width of the land wall is generally 6 to 10 ft, and wall thickness at lower elevations are governed by size of conduits and openings for operating facilities and by stability requirements.

Design of the river wall is limited by its location adjacent to the spillway. Spillway releases flow along the river face of the river wall, and that wall may be designed with uniform batter to provide smooth flow conditions. When the river bed is of erodible material, special protective measures (such as sheet piling or heavy stone protection) along the wall are required to prevent scour from undermining the wall. The river wall is primarily subject to hydrostatic loading, as with the water surface in the lock chamber at upper pool level and lower pool level in the river below the dam, or with the lock chamber unwatered for repair or inspection and lower pool level in the river below the dam.

In the case of two locks side by side, the intermediate wall has a constant top width, the same as required for the gate bay walls. Both faces of the intermediate wall (which form the sides of the two lock chambers) must have continuous straight surfaces for the tows to rub against as they pass through the lock and to provide smooth vertical surfaces for mooring during lockage. Thus, the upper portion of an intermediate wall cannot be narrowed for economy of construction.

Upper and Lower Gate-Bay Walls. These walls house the gate recesses, gate anchorages, gate machinery, and sometimes culvert valves and culvert bulkheads. The top of gate-bay walls must be sufficiently wide to:

a. House the operating mechanism.
b. Provide space for gate anchorages.
c. Enclose the valves.
d. Allow the gates to recess flush with the face of the wall for miter and sector gates.
e. Provide sufficient concrete between the culverts and gate recesses for stability.

Culvert-intake Walls. These walls extend immediately beyond the upper gate bays and provide space for the intake ports for the filling system. They are wide at the top to support:

a. Bulkhead-handling machinery when temporary closure structures are used.

b. Provide bulkhead recesses.
c. House floating-gage wells and other equipment.

Culvert-discharge Walls. These walls extend from the downstream end of the lower gate bay monoliths to the approach walls. They are usually lower than the lock chamber walls because they are below the lower gates and are subjected only to lower pool or high-water stages below the dam. They house the culvert-discharge manifold and diffuser system. When bulkheads are placed downstream from the discharge ports, the loads resisted by the culvert-discharge walls are similar to those on the lock walls during unwatered conditions.

Approach (Guard and Guide) Walls. Approach walls are extensions of the lock chamber walls at both ends of a lock and are required for all locks with barge traffic because tows have poor control and maneuverability when entering and leaving locks at low speed. Approach walls reduce hazards for tows entering and leaving the lock and reduce damage to both tows and lock facilities. They speed up lockages by offering a wider target for tows heading into a lock and provide temporary mooring space for tows with more barges than can be locked through in a single lockage or for tows queued for passing through the lock. Optimum alignment, length, and design of approach walls should be investigated in a general model study.

At locks used by both large ships and shallow draft tows, long guide walls can be an obstruction to the ships which cannot enter a lock under their own power, but must be moved into and out of the lock chamber by tugs or towing engines on lock walls (as at the Panama Canal locks).

One approach wall, the guide wall, is usually longer than the other, the guard wall. The guide wall serves to guide tows into the lock, and tows can put out lines to check posts on the wall to correct alignment for entering the lock. In the United States, many barges are 35 by 110 ft and are locked through three abreast (105 ft total width) in a 110-ft wide lock chamber, leaving little clearance along the lock walls. Guide walls are usually straight-line extensions of the lock chamber walls; however, where guide walls serve as mooring areas, the mooring reach of wall should be flared away from the approach or offset from it.

The shorter guard wall is designed to improve lock entrance and exit conditions for tows and to prevent tows from drifting into areas with hazardous currents and turbulence.

Guide walls can be located on either side of the lock approach, depending on site and current conditions, but are usually located along the landside. However, where cross currents exist in the upper approach because of spillway or powerplant operation or in the lower approach where a slow eddy often forms as the spillway or powerplant discharge widens out downstream of the lock, it is desirable to locate the guide wall on the river side and the shorter guard wall on the land side.

The usable length of the guide wall is usually equal to the length of the lock chamber; however, if the approach is well protected from wind and there are no adverse currents, a shorter length may be satisfactory. If conditions in the upper portion of the downstream approach are hazardous due to turbulence or high velocities, the usuable length of the lower guide should be measured from the point where velocities are less than 6 ft/sec or where excessive turbulence ends (Davis, 1989). Where banks are rock and tows cannot nose safely into the bank to queue for passage through the lock, it may be desirable to lengthen the guide wall to provide mooring space for more than one tow. In this case, the use of mooring piles should be considered, rather than longer walls, to reduce costs.

Approach walls (guard and guide walls) must be able to absorb impact and withstand abrasion from moving tows; however, local damage or failure of an approach wall when hit by a tow is not a serious matter because the lock can continue in operation while repairs are made. Various types of construction have been used for approach walls, each having advantages at particular sites:

a. Guard walls are either solid or are provided with openings (ports), depending on flow patterns and velocities at the specific site. For locks in reservoirs, the upstream approach walls may be slotted to avoid flow concentration, cross-currents, and high velocities at the upstream end of the walls.

b. Gravity walls have been used for approach walls on rock, soil, and pile foundations, but are expensive and rigid and require cofferdam protection during construction. In the United States most locks on rivers have concrete gravity walls. If rock is excavated to provide project depth in the lock approach, the wall can be placed on top of sound rock and the vertical rock face below the wall lined with concrete.

c. Reinforced concrete continuous walls are sometimes used, but they are expensive. Cofferdam protection is required during construction, and the thin sections are not as resistant to impact as are walls of other types.

d. Floating concrete guide walls have been used in the upstream approach at some locks in reservoirs where depths are large or foundation conditions are difficult.

e. Sheet pile construction (cantilevered or tied-back steel) can be used for landside approach walls where backfill extends to the top of the wall and where the approach channel is earth. The wall is set back from the face of the lock walls an amount equal to the thickness of timber fenders bolted to the piling. Construction cost is low, but such a wall can be severely damaged by impact of tows. Steel sheet piling in double rows, connected by diaphragms or tie rods and filled with earth can be used to form a continuous wall, and the top of the wall can be capped with concrete.

f. Cellular steel sheet piling filled with sand can be capped with concrete and supported by bearing piles. Reinforced concrete beams can be used between the cells to form a continuous rubbing surface for tows.

g. Isolated guide or mooring facilities, such as concrete piers, sheet pile cells, and timber-pile clumps equipped with tie-up equipment, may be used at the ends of approach walls to absorb much of the impact from a tow out of control and to serve as mooring points for tows waiting to lock through.

8.7 Lock Filling and Emptying Systems

The type of filling and emptying system used for a particular lock depends on the lift, tonnage capacity required, importance of the waterway, and construction costs. Lift is the most important factor. For low-lift locks (lifts less than 30 ft), a wall culvert-side port system can be used, but an intermediate-lift lock requires a more elaborate design, such as bottom lateral manifolds. For high-lift locks, it is usually desirable to use a bottom longitudinal manifold system that splits the flow vertically in the main wall culvert by means of a horizontal diaphragm and produces equal division of flow to four branch manifolds in the floor of each half of the lock chamber.

Design of modern locks in the United States, including design of filling and emptying systems, has been based on model studies, primarily by the Corps of Engineers and the Tennessee Valley Authority.

To minimize lockage time, lock filling and emptying systems should fill the lock chamber in the shortest practicable time without disturbances that would endanger vessels or the lock itself, particularly lock gates. Filling time is a function of lock lift. In the United States locks with miter gates with lifts of 30 ft or less have filling times of 6 to 8 minutes, and locks with lifts of 30 to 60 ft fill in 8 minutes. For higher lift locks (60 to 100 ft), filling time is greater than 10 minutes.

There are several different basic schemes for lock filling and emptying systems and numerous modifications of the basic designs for specific site conditions, as described later in this section.

Hawser Stresses. Two types of disturbances in lock chambers related to lock filling and emptying operations can be hazardous to tows being locked through:

a. Local turbulence generated by water entering or leaving the lock chamber and the lower lock approach.
b. Surging in the lock chamber as it is filled or emptied.

Tows and vessels in lock chambers are moored to the lock walls by hawser lines. Turbulence related to filling and emptying operations may damage small craft or individual barges in a lock, but surging is the more dangerous because it can cause an entire tow to break loose from the hawsers in the lock chamber and damage the lock, lock gates, or the tow itself. Stress in the hawsers is primarily a function of gross tonnage of the tow and slope of the water surface in the lock.

In the hydraulic design of locks, both longitudinal and transverse hawser stresses are measured in hydraulic models, but tows in a model are more closely restrained than in the prototype and there is more strain in the lines in the prototype than in the model. Thus, prototype stresses are normally less than measured in models, Figure 8.14. Measurements of hawser stresses in models and prototypes have been compared for many years, and it has been concluded that if prototype stresses measured in models do not exceed the following criteria a lock will be safe for barge tows and other vessels:

a. For various numbers and sizes of barges in a lock chamber, hawser stress should not exceed 5 tons and turbulence must not be hazardous for barges and small craft. The 5-ton value is the result of consensus reached in the late 1960s by tow operators and owners, lock operators, and laboratory and design engineers.

b. For single vessels up to 50,000 tons in a lock chamber, hawser stresses should not exceed ten tons.

c. For single vessels larger than 50,000 tons, hawser stresses are allowed to exceed ten tons since such vessels are restrained with more lines than tows or smaller vessels.

Summary data of permissible filling and emptying times for a 1200- by 110-ft lock to keep hawser stress within 4-, 5-, 6-, and 7-ton limits (for lock lifts of 20, 30, and 40 ft) are shown in Figures 8.15 and 8.16, respectively (Davis, 1989).

<u>Filling and Emptying Over, Between, or Around Lock Gates</u>. A tainter gate on the upper lock sill, Figure 8.13, can be used to supplement lock filling by other systems. As the tainter gate is lowered beneath the sill, water flows over the gate and into the lock chamber. However, filling is normally accomplished by a special filling system consisting of:

a. Intake ports upstream of the gate sill (or in the gate sill).
b. Wall culverts.
c. Laterals or ports in the lock chamber.

At St. Anthony Falls locks on the Upper Mississippi River, tainter gates at the locks serve primarily as a supplementary spillway at flood stages and to pass ice and debris through the lock.

Sector gates, Figure 8.13, are used as lock gates where reversal of head of significant duration occurs, for example at a location affected by tidal action where the downstream water surface is sometimes higher and sometimes lower than the upstream pool. As the sector gates swing apart, water flows into or out of the lock through the opening between the gates, and the lock chamber must be sufficiently long so that tows in the lock can be safely moored beyond the region of local turbulent inflow. Some sector gates have been designed to also admit water around the gates through the wall recesses.

Sector gates can be used with heads up to about 20 ft, and reversal of head rarely occurs at locks with normal lifts greater than 20 ft. Although sector gates are designed to operate at the estimated maximum lift, such conditions are usually of short duration and relatively infrequent; normal lifts are usually much less. Sector gates are used only when required, because other types of gate are usually more economical to construct and other types of filling systems provide more satisfactory operation.

<u>Filling and Emptying by Valves in Gates or through Short Culverts</u>. Early locks in the eastern United States used valves located in the lock service gates or short culverts through the river wall (each controlled by a separate valve). However, modern designs use stub or loop culvert systems around the service gate, Figure 8.17a. In this design, short culverts in the service

gate monoliths carry water from the upper pool, around the gate, and discharge it into the lock chamber immediately downstream of the gate. Such systems are most economical where a lock is excavated in rock and walls are too thin to accommodate wall culverts. Systems of this type are also used to empty a lock, Figure 8.32a.

<u>Filling and Emptying through Wall Culverts and Ports or Laterals</u>.

- **Early conventional wall culvert and port systems.** The following systems were widely used for early locks on the Ohio, Tennessee, and Upper Mississippi Rivers and have performed well for low lifts:

a. Wall intakes in the upper approach walls.
b. Longitudinal culverts in the lock walls.
c. Wall filling and emptying ports throughout the length of the lock chamber.
d. Wall discharge system downstream of the lower lock gates.

At some locks incremental valve openings have been used to reduce turbulence in the lock chamber when filling and in the lower lock approach when emptying. Ten of the 11 locks initially constructed by the Tennessee Valley Authority (TVA) with conventional systems have comparatively high lifts of 39 to 80 ft, and at some locks the valve operating time is lengthened by holding the valve in a partly open position for various periods, depending on the size of tow being locked through. Turbulence and transverse and longitudinal currents occur to varying degree at locks on the Upper Mississippi River constructed in the 1930s, and valve opening times are lengthened to improve navigation conditions.

- **Modern systems.** In an effort to lessen problems experienced with turbulence and currents using the conventional culvert and port design, more complex systems were developed to provide faster and safe filler and emptying operations. Modern systems for locks of low and medium lift are generally of two types, and which system is used at a particular site is influenced by foundation materials and traffic and is ultimately determined by economics.

a. Systems filling and emptying the lock chamber through ports along the base of the lock walls (side wall port locks or side port locks), Figure 8.18a. This is the most common type of lock on the inland waterway system in the United States and can be used for lifts from 5 to 30 or 40 ft depending on lock chamber size, but generally is not suitable for higher lifts.
b. Systems filling and emptying the lock chamber through laterals and ports or longitudinal culverts and ports recessed in the floor of the lock chamber, Figures 8.18b and 8.19.

These systems take water from the upper pool through an intake manifold into wall culverts that supply water to ports in the lock chamber. The lock is usually emptied through the same system of ports and culverts and through a discharge manifold that discharges water either into the lower lock approach or riverward of the river wall of the lock.

Systems filling and emptying the lock through ports along the base of the lock walls operate satisfactorily with moderate filling times, with the time required for filling dependent on the lift and size of the lock chamber.

Filling and emptying systems recessed in the lock floor (laterals with ports for low-lift locks, Figure 8.18b, or longitudinal culverts with ports for high-lift locks, Figure 8.19) are designed for fast filling times; however, they require deeper excavation than is required for locks with ports along the lock walls. If the excavation is in rock, the additional cost may be hard to justify. If traffic can be served safely by a lock with moderate filling time, such a design is the cheapest and best solution. However, if projected traffic requires so many lockages that a fast-filling system or a second lock would be required, use of a more complex fast-filling design is usually the cheapest and best solution.

Davis (1989) presented data relating lock volume to average filling inflow, Figure 8.20. These curves can be used to obtain a preliminary estimate of lock filling filling time.

- **Intake manifolds**. Intake manifolds consist of a series of ports opening into a larger area that transitions downstream into a smaller rectangular cross section at the culvert control valve, Figure 8.21. The use of multiple ports spreads the incoming flow over a larger area than if a single large port were used, and this reduces the formation of vortices and entrainment of air into the wall culverts.

Intake manifolds are usually located in approach walls, but in some cases are in the upper gate sill, as shown in Figure 8.21, to pass drift and ice through the lock or to provide supplementary discharge capacity. Intake manifolds are streamlined and are designed for flow in one direction only, and intake velocities are usually limited to about 8 to 10 ft/sec. All ports are the same size at the wall face, but have different throat dimensions. The height/width ratio of ports at the wall face is usually in the order of from 2:1 to 4:1. The total port area at the wall face is about 2.5 to 3.5 times larger than the culvert cross-sectional area to reduce intake velocities and thus:

a. Reduce intake losses.
b. Minimize the formation of vortices that draw air into the system and create turbulence in the lock chamber when the air is discharged through the ports.
c. Minimize damage to trash racks on the intake ports.

The throat area of each port in the intake manifold is decreased successively in a downstream direction to obtain equal flow distribution through all ports. The head loss coefficient for the intake manifold is a function of the ratio of total port throat area (ΣA_p) to culvert area (A_c) and decreases as the ratio increases, Figure 8.22. A value in the order of 1.8 is desirable; values ranging from 1.5 to 2.0 have been used successfully (Davis, 1989). Comparison of model and prototype data shown on Figure 8.22 indicates that further increase in the ratio of $\Sigma A_p/A_c$ beyond a value of about 2 has minimal effect on the head loss coefficient. Head loss through the intake manifold can range from 0.16 to $0.4\ V_c^2$ where V_c is culvert velocity.

Much shorter culvert intake walls are required if intake manifold ports can be located on both faces of the walls (Siamese intakes), as for Barkley Lock, Figure 8.17b.

The top of intake ports should be located well below the minimum upper pool level to ensure positive pressure in the system. Davis (1989) suggests that minimum submergence below the minimum upper pool level be set equal to the velocity head at the throat of the most downstream intake port.

Trash racks are used on the face of intake ports, Figure 8.23, to prevent debris and ice from being drawn into the system. When floating drift or ice is present, it is important that intake velocities be limited to 8 to 10 ft/sec to avoid impact damage to the trash racks. Slots are provided in the lock walls for the installation of bulkheads for unwatering the intake area for inspection and repair.

Vortex action and entrainment of air at intake ports in gate sills can:

a. Reduce efficiency of the filling system.
b. Present hazards to operating personnel and small craft.
c. Produce dangerous conditions in the lock chamber when large blocks of air are expelled through the filling ports.
d. Result in damage to trash racks by debris caught in the vortex.

In general, vortex action has been found to be greater in the prototype than in models. Model studies and prototype experience have shown that intakes in the upper gate sill are more susceptible to vortex action than are intakes in the lock walls. At sill intakes there are concentrations of high velocity in the approach and in the port entrances because the width of flow is restricted to that of the lock sill; the closed angle of miter gates affects uniformity of the approach flow; and discontinuities at miter gate recesses can induce eddies leading to the formation of vortices.

In model studies of intake manifolds located on the top of gate sills parallel to the upstream gates, as at the St. Anthony Falls Lower Lock, Figure 8.21c vortex action was reduced or eliminated by:

a. Decreasing the distance between the intake manifold and upper lock gates.
b. Increasing the spacing between intake ports.
c. Increasing the intake port area at the sill face.
d. Increasing submergence of the intake.

To reduce vortex problems at both wall and sill intakes, Davis (1989) recommends avoidance of the following conditions:

a. Unequal distribution of flow in the intake ports.
b. Openings in the guide or guard walls that induce diagonal currents.
c. Breaks in alignment of the approach walls.

Small vortices carry little or no air into the culverts and have essentially no effect on lock chamber turbulence; however, large vortices can produce considerable turbulence. Vortices are difficult to avoid in high-lift locks for shallow-draft traffic where the depth above the upper sill is shallow and the approach floor is at about the elevation of the upper sill.

There are no design criteria that will ensure that lock approaches will be free of vortex problems, but the problems will be minimized if flow conditions are symmetrical, velocities are minimized, and maximum submergence is provided. Where a problem is anticipated, it should be investigated in a hydraulic model.

One of the more recent model studies to investigate potential vorticity at a lock intake was a study at the Waterways Experiment Station of conditions at replacement locks at the old Gallipolis Locks and Dam on the Ohio River (Davidson, 1987). The two new locks (110- by 1200-ft and 110- by 600-ft) are located in a short excavated channel across the inside of a bend. Two alternative intake designs were considered, Figure 8.24. Intake designs were tested on a 1:25 scale model that reproduced 2500 ft of the Ohio River beginning 188 ft upstream of the existing lock guide wall. Model studies indicated vortex problems would occur with both intake schemes as originally designed. However, modifications developed in the model eliminated vortex formation for both designs.

Alternative I, Figure 8.24a, involved filling the locks from the river through three long culverts. In testing, it was observed that flow conditions were unsatisfactory and that the following contributed to the formation of severe air-entraining vortices at the intake structure:

a. Flow entering the intake was unsymmetrical.
b. Layout of the original approach walls caused water to swirl around the abutments.
c. There was insufficient submergence for the design.

Modifying the position of the approach wall to Position 2, Figure 8.25a, decreased the severity of the vortices; however vorticies still occurred. Various other modification were studied. The invert of the intake was lowered 15 ft, and flow entering the intake was made more symmetrical by relocating the approach walls and placing a dike upstream of the existing lock guide walls. A vortex suppressor plate 15.4 ft thick was placed at the same elevation as the intake conduit roof and extended 17 ft upstream to the trash rack, Figure 8-25b. With these modifications, all air entraining vortices were eliminated.

Alternative II, Figure 8.24b, involved filling the locks from the river through a short excavated channel supplying water to two intake manifolds in the guide wall and a third manifold in an intake tower. The original design of the intake tower had a sharp corner at the upstream front face, and a severe vortex developed at that point; however, no vortices formed at the manifolds in the guide wall. Several modifications of the intake tower were tested. A design adding a straight vertical wall with a quarter of an ellipse immediately upstream of the intake tower, Figure 8.25c, eliminated air-entraining vortices for this alternative.

- **Control valves.** Control valve in lock culverts are usually tainter gates in a "reverse" position, that is, with the trunnions on the upstream side and skinplate and sill on the downstream side, as shown schematically in Figure 8.26. With two exceptions, all locks built in the United States since 1940 have reverse tainter control valves. Positioning the valves in this manner prevents air entrainment in the low pressure area downstream of the valve, thus minimizing turbulence and high hawser stresses associated with release of air from the filling system into the lock chamber. Air entrainment becomes a more severe problem as lock lift increases.

Lock filling criteria is based on not exceeding permissible hawser forces of 5 tons. Hawser stress is related to turbulence which, in turn, is related to the depth of water (cushion) in the lock chamber and over the filling ports. The cushion provided has a major impact on project costs, and the depth normally is not greater than needed for bottom clearance (in the order of a few feet) because of cost. Depending on specific site conditions, such as depth to sound rock, it is sometimes economical to provide greater cushion.

Recommended prototype valve opening time to limit hawser forces to five tons for lock chambers of various sizes, based on model studies and reported by Murphy (1975), are shown in Figures 8.27 and 8.28.

- **Wall culverts**: Culverts in lock walls convey water from the intake manifold to the filling and emptying system, and to the outlet system. Downstream from the intake manifold, the culvert transitions to a rectangular or square section at the filling valve, with a culvert height to width ratio of from about 1.0 to 1.15. In wall culvert side-port systems, the culverts are usually of uniform size from the filling valve to the emptying valve. Any culvert expansions should be gradual, about 1 on 10, to minimize head loss and turbulence.

The horizontal location of a culvert in the lock wall, the distance from the culvert to the face of the lock wall, fixes the length of wall ports. This distance is sometimes determined by structural requirements. As a minimum, a port length of about 8 ft is desirable. In side-port systems, the elevation of wall culverts is established by submergence requirements for the ports and pressure conditions at the valves. In bottom filling systems, minimum depth in the lock chamber must also be considered. In high-lift locks with bottom longitudinal systems, valves are placed low to control pressures and air intake, and the valves and wall culverts must be almost as low as the bottom manifold system.

To unwater valves for maintenance, bulkhead slots are usually placed in culverts upstream and downstream of the valve. To minimize cavitation damage, the downstream slot is located downstream of the vena contracta when the valve is 50 to 70 percent open; locating the slot a distance of three times the culvert height downstream will usually place it out of the area most susceptible to cavitation. The upstream bulkhead slot is located at least two times the culvert height upstream of the upstream edge of the valve shaft. For high-lift locks, steel plate culvert liners are used on all surfaces of the culvert downstream from the valve. Model tests indicate that the area most subject to cavitation damage is usually 2.0 to 2.5 times the culvert height downstream from the bottom seal line of the reverse tainter valve, and the liner extends downstream past this area.

- **Wall-port systems**. Wall ports to fill and empty a lock chamber are designed for flow in both directions. They are streamlined, with rounded entrances and exits, and are flared to the lock face to reduce exit velocities when filling. The design shown in Figure 8.29 is considered the best of many designs tested. The ports occupy 50 to 60 percent of the lock length and are located in the center portion of the lock chamber to minimize surging during filling. Ports in one wall are staggered with respect to ports in the opposite wall so that the jets from one wall do not collide with jets from the opposite wall, but pass each other and there is good distribution of energy with little turbulence.

As the filling jet exits the port, it flares upward at about 7 degrees; thus flaring about 14 ft when it reaches the opposite wall of a 110-ft lock. If wall ports are staggered and set on 28-ft centers in a 110-ft lock (at 20-ft spacing in an 84-ft lock), this expansion takes place between jets issuing on the opposite wall, minimizing turbulence. Culverts and valves should be sized to carry the jets to the far side of the lock chamber, but port outflow should not be sufficient to cause a welling up of water on the far side of the lock.. Locks narrower than 110 feet have side ports set on lesser spacing and lesser discharge from the wall ports to avoid upwelling on the far side.

Wall ports should be of sufficient size so that the jets do not completely diffuse before reaching the opposite wall or boils will occur at the surface, thus increasing hawser stresses. For higher lock lifts, the ports may be directed down toward the base of the opposite wall to reduce turbulence. Model studies for Arkansas River locks indicated that triangular recesses, Figure 8.30, in front of the upstream one-third of the ports would reduce upstream longitudinal hawser stresses during filling.

For shallow-draft locks, the bottom of wall ports should be set at the elevation of the bottom of the wall culvert and at, or slightly below, the level of the lock floor.

The total port throat area in one wall should be about 95 percent of the wall culvert area. A smaller ratio would increase filling time, and a larger ratio would result in less favorable hydraulic conditions in the lock chamber. Port face area varies with lock chamber size, as recommended by Davis (1989):

a. 10 to 11 sq ft for a 1200- by 110-ft lock.
b. 9 to 10 sq ft for a 600- by 110-ft lock.
c. 6 to 7 sq ft for a 600- by 84-ft lock.

- **TVA multiport system.** In 1959 the TVA used a somewhat different filling and emptying system in design of three new locks with lifts of from 42 to 60 ft on relative high rock foundations. The multiport system, Figure 8.31, required less excavation and was more economical than the culvert-lateral-port system.

- **Lock chamber lateral diffusers.** A lock chamber lateral diffuser is a filling and emptying system of small culverts (laterals) across the lock chamber with ports in the laterals. The laterals are recessed in a trench in the lock floor so that the jets mix and dissipate most of the energy below the main body of water and tows in the lock chamber, minimizing hawser stresses. A typical installation is shown in Figure 8.18b. They are more expensive than wall port systems, but may be economically justified at some locks serving heavy traffic on the basis of reduction in lock filling and emptying times.

Lock chamber lateral diffusers are similar to discharge diffusers, but differ in that they are designed for flow in both directions, that is for both filling and emptying operations. They are located in the middle third of the chamber for a 600-ft lock, as for the Greenup auxiliary lock, Figure 8.18b. For a long lock, for example the 1200-ft Greenup main lock, Figure 8.18b, the diffusers are split into two systems to keep hawser forces within acceptable limits and one group of laterals is located approximately the middle third of the upstream half of the chamber and the other in the middle third of the downstream half of the chamber.

- **Lock emptying systems.** Lock emptying systems are designed to discharge and distribute outflow from lock emptying so as not to cause turbulence or currents that would endanger craft in the lower lock approach. Outlet systems usually discharge either to the lower lock approach (between the lower guard and guide walls) through wall port manifolds or laterals, or on the river side of the lock, Figure 8.32. The emptying culvert is widened downstream of the emptying valve to reduce exit velocities and head loss, and the discharge system is designed for flow in one direction only.

Where locks empty into the lower approach, a system of laterals across the lock usually is used to minimize turbulence. The single culvert discharge laterals for St. Anthony Falls Lower Lock, Mississippi River, is shown in Figure 8.33. The flow area (cross section) of the laterals is decreased in the downstream direction (across the lock) at successive ports for uniform discharge through the ports. The outside walls of the laterals are parallel, and ports in adjacent laterals are staggered so that jets issuing from the ports are offset and do not collide and can diffuse laterally before reaching the opposite wall.

The emptying system for the Snell Lock, St. Lawrence Seaway, Figure 8.32b, has discharge culverts in both walls and extensions on all ports to direct the jets perpendicularly across the trenches and produce a better flow distribution in the lower approach.

Wall discharge manifolds have been designed to empty completely or partially into the lower lock approach, as for the McArthur Lock, St. Mary's River, Figure 8.32a, and the New Cumberland Main Lock, Ohio River, Figure 8.34. The New Cumberland emptying system was designed to divert two-thirds of the discharge outside the lock approach. Ports discharging into the lock approach are staggered to minimize interference by opposing jets.

Davis (1989) reports that turbulence experienced at these three prototype locks is greatest for the St. Anthony Falls Lower Lock, less for the New Cumberland Lock, and least for Snell Lock. However, cushion depths over the outlets varies significantly for these locks, being 22.2 ft at St. Anthony Falls, 24 ft at New Cumberland, and 48 ft at Snell.

Discharge into the lower lock approach during emptying can create currents that adversely affect upbound tows. When the emptying manifold is placed riverward of the lock, the emptying operation generally has no effect on tows approaching the lock. The emptying system for Greenup Locks, Ohio River, Figure 8.32c, is typical of systems that divert the entire outflow riverward of the lock. Stilling basins are usually included in such outlets to reduce turbulence. With such designs, the lower lock entrance is completely free of disturbances during emptying operation and the entire length of the guide wall can be used for mooring tows. However, outlets such as for Greenup Auxiliary Lock may cause a problem with miter gate operation at moderate flows when the stage in the lock approach is lower than at the outlet (and in the lock chamber) causing a head differential at the gates. Miter gates normally require equal water levels on both side of the gates for opening.

Another outlet design, used for some of the Arkansas River locks, includes a system of baffles, Figure 8.35. Such designs are suitable for low-lift locks at some locations.

A recent example of an emptying system discharging on the riverward side of the lock is the Olmsted project now (1995) under construction on the Ohio River 16.6 miles upstream from the junction of the Ohio and Mississippi Rivers. The Olmsted project, with two 110- by 1200-ft locks and a design lift of 21 feet, will replace two existing locks Locks and Dams 52 and 53, having lower lifts. Tailwater at the site is not affected by a downstream navigation structure. Open-river conditions prevail downstream, and tailwater is influenced by Mississippi River stages. The emptying systems are unique in that discharge culverts from the land wall, the middle wall, and the river wall all empty into a common outlet structure in the river, and culverts from the land wall and middle wall pass under the floor of the river lock (Stockstill, 1992). The outlet is located 25 ft riverward of the river lock in the vicinity of the lower gate monolith. The 14- by 18-ft culverts drop 21 ft vertically over a 76-ft length in the lock walls and then turn 90° to the outlet, Figure 8.36.

Bottom Longitudinal Filling and Emptying Systems. For higher-lift locks, the bottom longitudinal filling and emptying system, with longitudinal culverts with ports recessed in the floor of the lock chamber, has become widely used, as for Lower Granite Lock on the Snake River, with a 32-ft lift, Figure 8.19. These systems are complex in design, but model studies indicate they are superior to other systems for medium- and high-lift locks because of low turbulence in the lock chamber and low hawser stresses, with less chance of damage to tows or to the lock itself. In the bottom longitudinal system, flow in the wall culverts passes into a "crossover" culvert across the lock at the center of the lock chamber, as for Dardanelle Lock on the Arkansas River, with a 54-ft lift, Figure 8.37. A splitter wall in the crossover culvert distributes flow equally to two longitudinal floor culverts with ports, one in the upstream half of the lock chamber and the other in the downstream half of the lock chamber.

The bottom longitudinal filling and emptying system for Dardanelle Lock on the Arkansas River, a "side-by-side" system, Figure 8.37, is representative of such systems designed in the 1960s. The design was later refined and modified, particularly for locks of higher lift and 1200-ft length. Murphy (1980) recommended that the side-by-side design not be used for lifts in excess of 60 ft based on experience with the Bankhead Lock on the Black Warrior River with a 69-ft lift, Figure 8.38.

Model tests indicated that the side-by-side system designed for Dardanelle Lock could fill the lock in 8.4 minutes with a maximum longitudinal hawser stress of about 5.2 tons with normal 2-minute valve operation. Model studies also indicated that baffles along the walls and between the longitudinal culverts, Figure 8.37b, would reduce bottom water movement toward the ends of the the lock chamber, reducing individual boils and turbulence so that conditions would be satisfactory in the lock chamber with normal operation.

Examples of the "over-and-under" bottom longitudinal filling and emptying system are shown in Figures 8.38 and 8.39. At both locks flow from the crossover culvert is directed to combining culverts upstream and downstream of the crossover culvert. At Lower Granite Lock on the Snake River (lift 105 ft), there are four floor culverts in each half of the lock chamber, while Bay Springs Lock on the Tennessee-Tombigbee Waterway (lift 84 ft) has two culverts in each half, Figure 8.39. The Bay Springs system under construction (looking upstream) is shown in Figure 8.40. The floor culverts are 14 ft wide and 9 ft high, each with 12 pair of ports 1.5 ft wide and 3.5 ft high, spaced 15 ft on centers, with a port/culvert area ratio of 1.0. With this design and a 1-minute valve operating time, the lock filled in the model in 9.9 minutes with longitudinal hawser stress of about 7 tons and transverse hawser stress of about 6.5 tons. The lock emptied in about 11.7 minutes.

- **Culvert area ratios**. Murphy (1980) suggested that a relatively constant cross-sectional area be maintained from the wall culverts through the crossover culverts and the combining culvert. He further suggested that initial studies of filling time and cost be primarily concerned with culvert size in this area and that filling valve size be determined later.

- **Longitudinal floor distribution culverts**. Murphy (1980) noted that in the 670-ft Bankhead and Bay Springs locks two distribution culverts in each half of the lock chamber were adequate, but that general tests of a 1200-ft lock indicated four were required in each half. He suggested that the number needed probably depends on lift and culvert size as well as on the length/width ratio of the lock chamber.

- **Port manifolds**. Murphy (1980) recommended that:

a. Port manifolds extend over at least 50 percent of the length of the chamber.

b. If two culverts are used in each half, manifolds be centered on the one- and three-quarter points of the chamber, with each manifold extending over at least 25 percent of the total length of lock.

c. If four culverts are used in each half, manifolds be centered on the one-, three-, five-, and seven-eights points of the chamber, with each manifold extending over at least 12.5 percent of the total lock chamber length.

- **Ports**. Ports tested in model studies have ranged in size from 4.2 to 6.28 sq ft, and Murphy (1980) favors a port similar to that used at Bay Springs (3.5 ft high, 1.5 ft wide, 5.25 sq ft total area) because those ports gave good distribution of turbulence in the lock chamber and are large enough to allow access for inspection and maintenance. He noted that, while in a sidewall port system the total cross-sectional area of ports should be about 95 percent of the culvert area, with the relatively short distribution culverts in this system a port-to-distribution culvert area ratio of 1.0 is preferable and that all available space should be used for the port manifold. He suggested there should be a relationship between trench size and port size, with lift also a factor, so that a large portion of the kinetic energy of jets from the ports is dissipated in turbulence in the trenches along the distribution culverts. Baffles are needed on the walls of the trenches and between the distribution culverts to prevent upwelling of jets from the ports.

- **Operation.** Due to differences in friction factors between model and prototype, prototype locks with bottom longitudinal culvert filling and emptying systems can be expected to fill about 16 percent faster than indicated by a 1:25-scale model (Murphy, 1980).

8.8 Closure Facilities for Locks.

All Corps of Engineers locks have facilities that can be set in place in still water for maintenance of the lock chamber and lock gates. However, few locks have the capability to make closures in flowing water under emergency conditions.

Navigation locks are vulnerable to accidents that result in damage and failure of lock gates so that the pool is drained down through the lock to the top of the upper gate sill. In the United States, accidents with tows ramming miter gates occur from time to time. In a typical case, a tow entering a lock rams and knocks out the closed gates at the far end of the lock chamber before gates behind the tow can be closed. (In one instance, a vessel out of control knocked out the gates at both ends of a lock.)

When miter gates are damaged and cannot be closed, uncontrolled flow through the lock chamber can result in significant losses. The extent of such losses depends principally on development upsteam and downstream of the lock. In a highly developed area, such as along the middle reach of the Ohio River, monetary losses and other hazards can result from:

a. Loss of the upstream pool.
b. Flood damage downstream from the lock.
c. Losses to shipping using both pools, particularly in the upstream pool.

Loss of the upstream pool storage can result in loss of municipal and domestic water supply if the water surface falls below the elevation of the water supply intakes, loss of condenser water for power plants, and losses and damage to tows and vessels that are beached on the channel bottom. Unrestricted flow through a lock may cause a sudden rise in downstream pool level, causing small craft and barges to break their moorings and drift uncontrolled into the channel, sometimes lodging against the spillway of the next dam downstream. A sudden and unexpected rapid increase in river stage can result in greater flood damage to equipment and installations than would occur with a normal slower river rise.

- **Maintenance closures.** Maintenance and operation costs should be considered in selecting the type of closure facilities for a particular lock. At a high-lift lock with a high upstream gate sill, a submergible vertical-lift gate or tainter gate at the downstream edge of the upper gate sill is generally the best solution. However, use of use of such gates at a low-lift lock could result in high maintenance costs because of sediment accumulation on the gate and the need for periodic costly sediment removal. Emergency closure structures can also be used for routine maintenance work that requires dewatering.

Maintenance closure facilities at most Arkansas River locks consist of a center post that is set in a recess in the lock sill with 55-ft long stoplogs on both sides extending over to the lock walls.

- **Emergency closures.** Various types of emergency closure structures have been used successfully. All have advantages and disadvantages, depending on local conditions:

a. Submergible vertical-lift and submergible tainter gates can be operated quickly under flowing water conditions. They can be used at locks with sufficient lift to allow the gates to be submerged downstream from the upper gate sill and above the lock floor.

b. Stop logs can be placed with a crane and hoist or with an overhead locomotive crane, the only difference in the installations being in the equipment used for placement. A crane and hoist can be used under any condition, and stop logs are the least costly type of emergency closure. An overhead locomotive crane has been used for placement of stop logs at some of the newer Ohio River locks; the overhead bridge on the gated spillway piers continues over the upstream end of the lock, and the same crane used to operate the spillway gates is used to place the stop logs in recesses in the lock walls. The operating bridge for the locomotive crane must be high enough to provide the vertical overhead clearance at the lock required for navigation (55 ft above the 2 percent duration flow on the Ohio River), and this may involve added costs for raising the spillway piers.

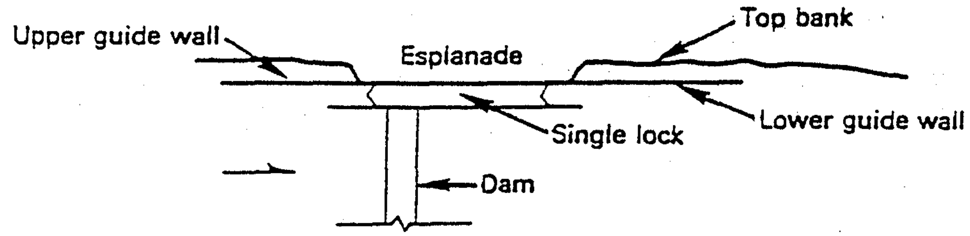
Stop logs placed with a derrick or crane have several advantages in addition to not requiring an overhead structure, including: reliability; no permanently submerged structures to maintain; and little maintenance required for the stop logs, hoists, and derrick or crane. Difficulties are that installing stop logs requires considerable time and space is needed for storage of the stop logs near the upstream end of the lock.

Stops logs cannot be placed individually in flowing water; water flowing over and under an individual stop log produces vortex trails that cause eratic movement, and the stop log jams in the wall recesses. Accordingly, the first stop log is placed in the recesses above the flowing water and held there temporarily; additional stop logs are added one by one, and the entire unit is lowered into the water incrementally, as each stop log is added.

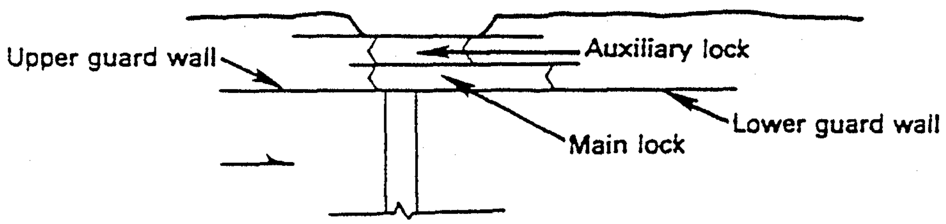
c. Sector gates can be closed in flowing water in a few minutes and require no special equipment, personnel, or mobilization. However, they are large structures and require considerable space, have a very high first cost, require maintenance, and may have a problem with differential settlement if the lock is wide.

d. Overhead vertical lift gates require a high structure across the lock and are unsuitable where high clearance is required for navigation.

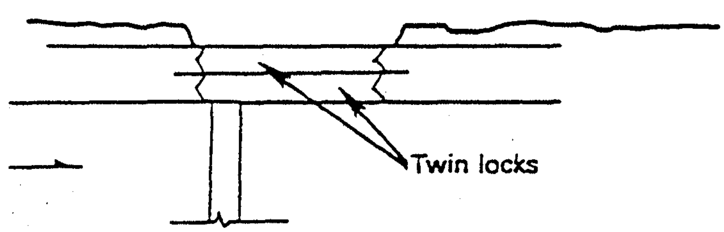
Emergency closure sills are discussed earlier in this section. Closure facilities, including all appurtenant equipment (cranes, hoists, trucks, and auxiliary power source), should be readily available and should be inspected periodically to ensure they are operable. Such facilities should be designed to close off uncontrolled flow as quickly as possible, depending on site specific conditions. At some locations (e.g. the Ohio River) uncontrolled flow should be stopped in 2 to 3 hours. The time factor may not be as critical at other, less-developed, locations.



(a) Layout for a single lock



(b) Layout for adjacent locks



(c) Layout for twin locks

Figure 8.1 Typical lock layouts.

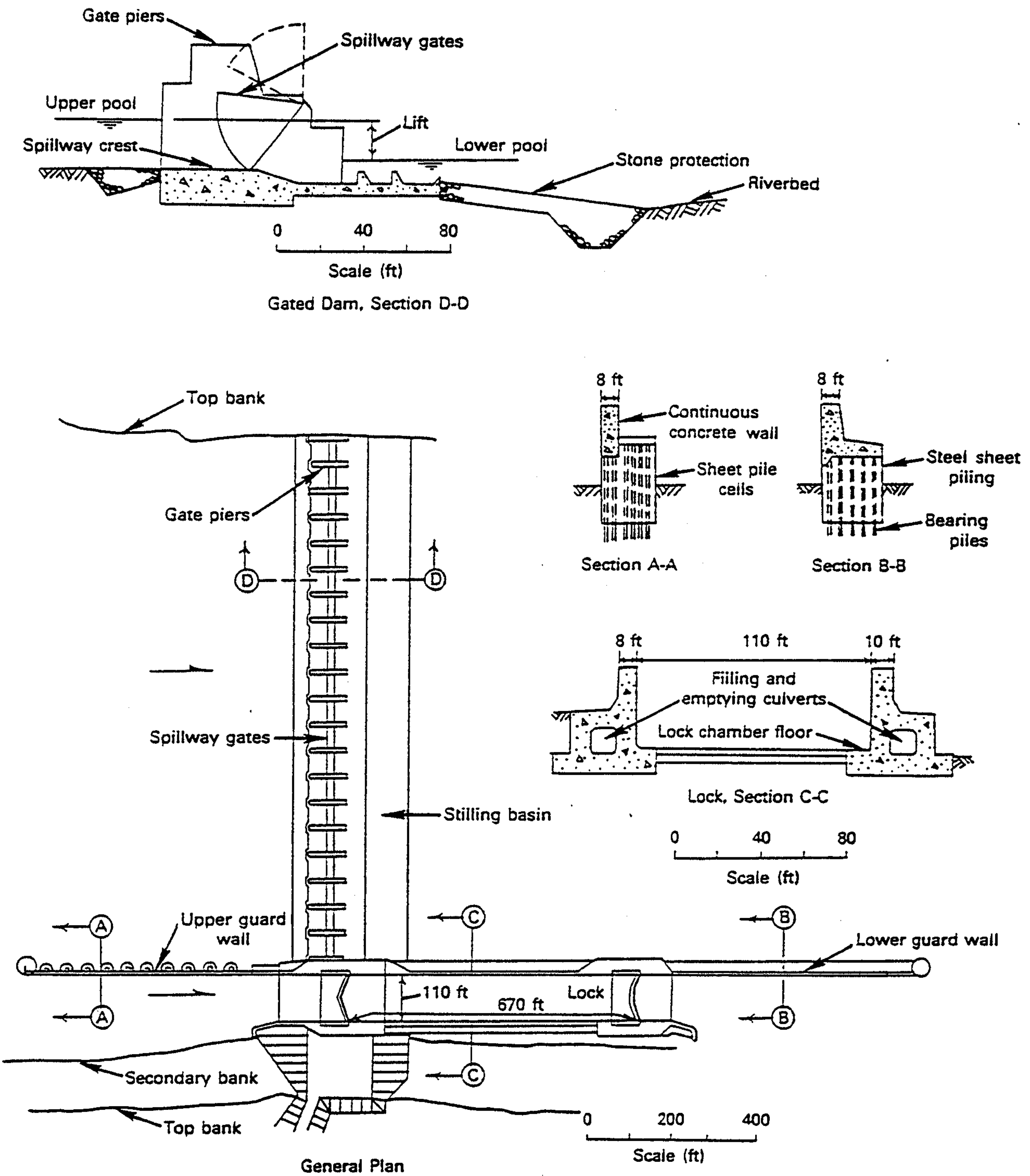


Figure 8.2 Typical low-lift navigation lock and dam (Ables and Boyd, 1966).

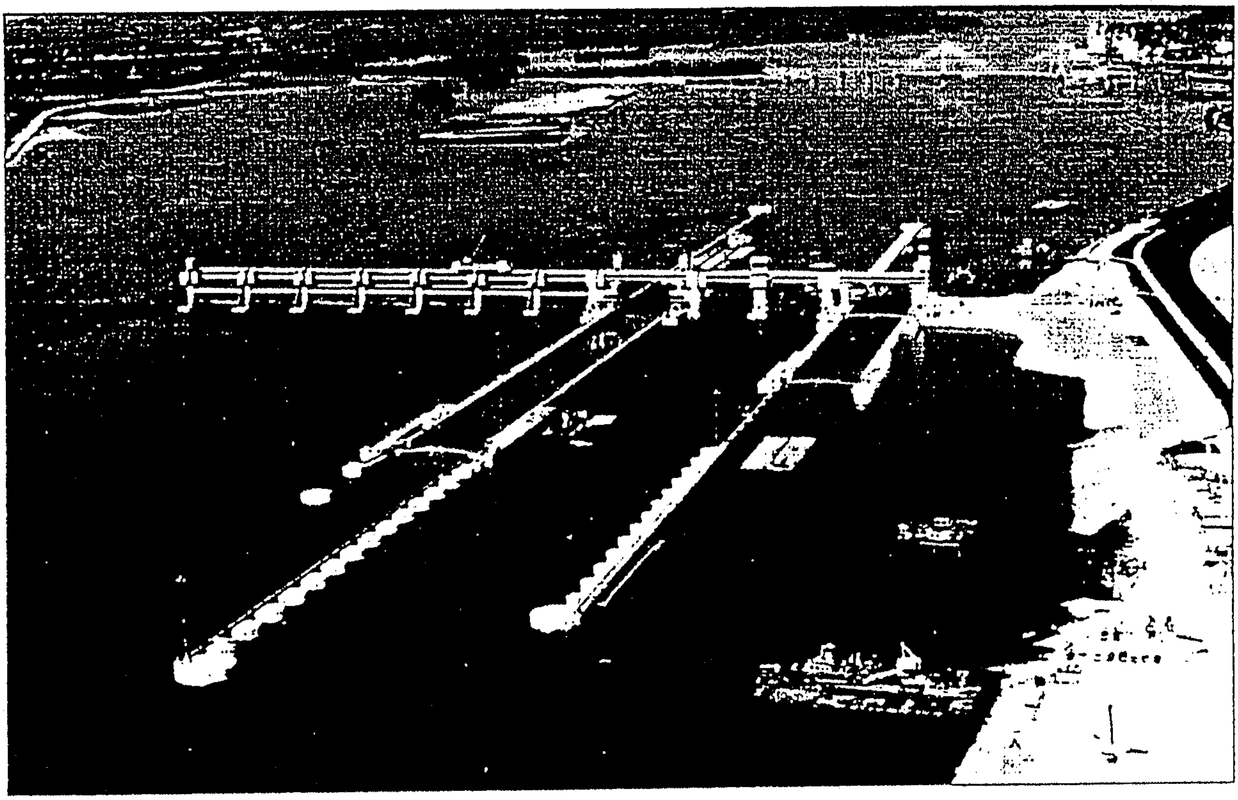


Figure 8.3. Melvin Price Locks and Dam, Mississippi River (Replacement for Lock and Dam 26).

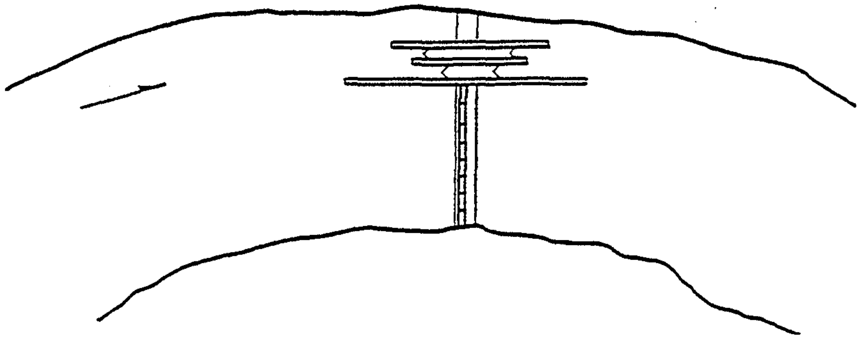
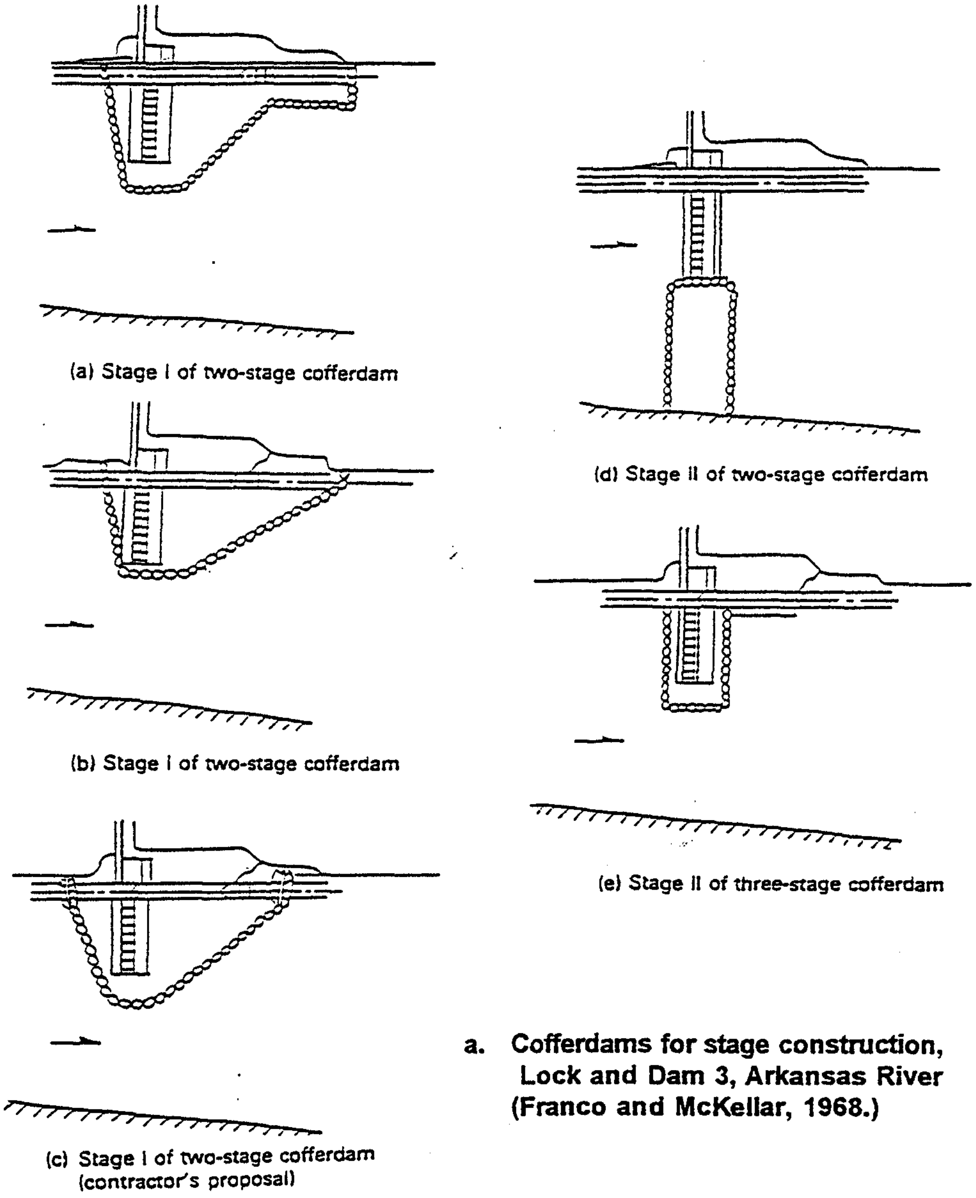
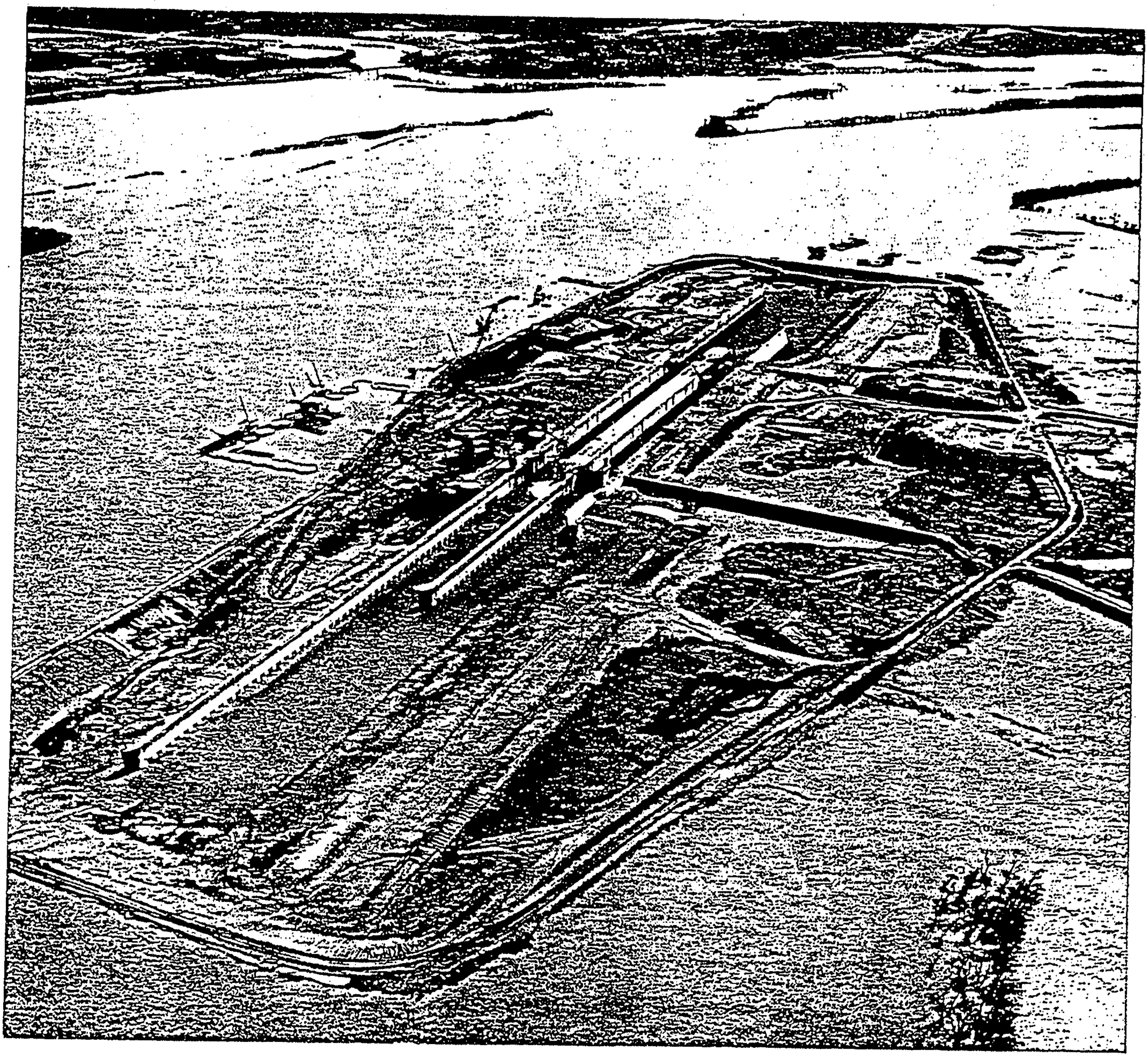


Figure 8.4. Lock in deep part of bend.



a. Cofferdams for stage construction, Lock and Dam 3, Arkansas River (Franco and McKellar, 1968.)

Figure 8.5. Cofferdams for stage construction.



b. Twin 1200-ft locks under construction, Ohio River, Smithland, KY.

Figure 8.5. Cofferdams for stage construction.

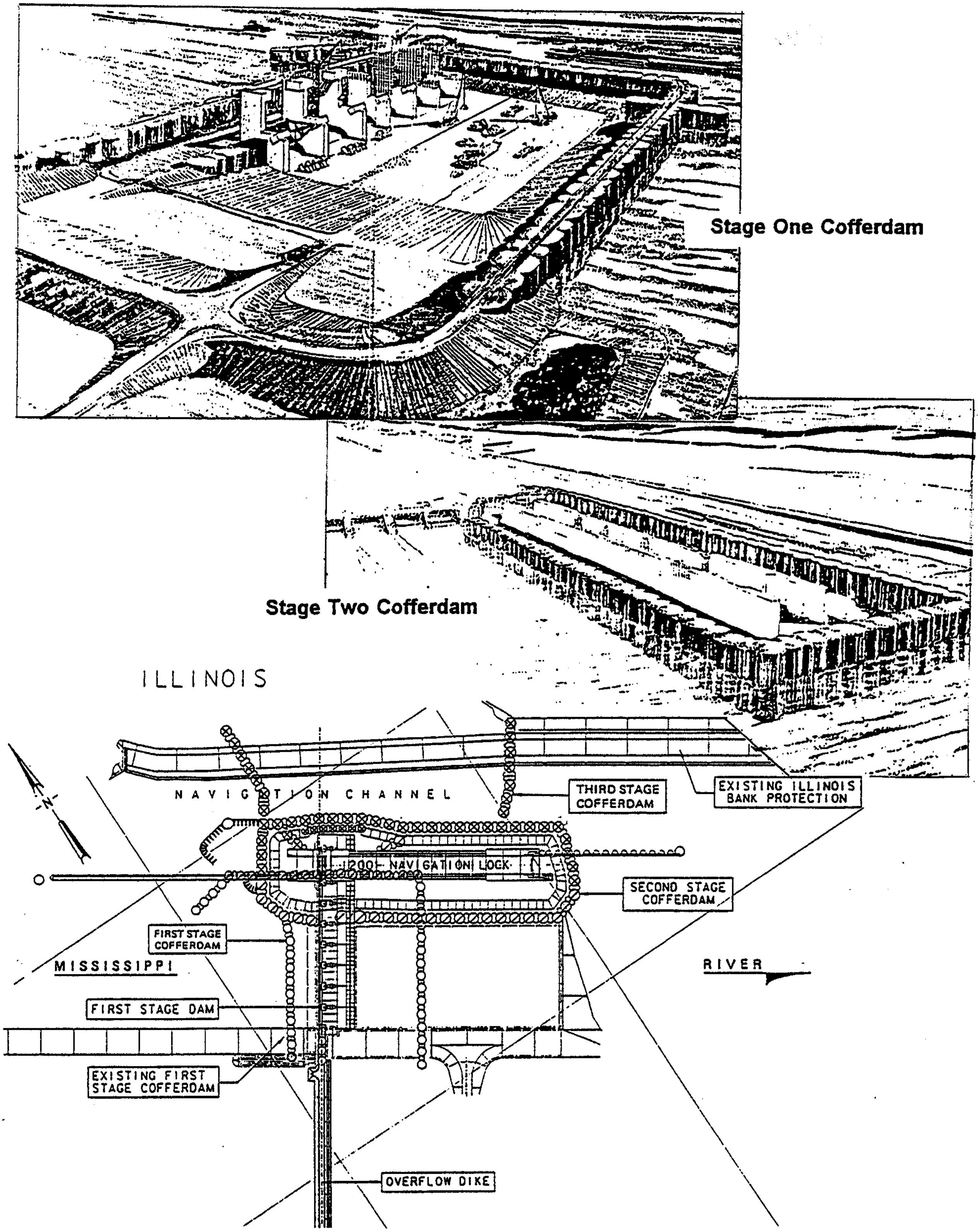


Figure 8.6. Three-stage cofferdam scheme, Replacement for Lock and Dam 26, Mississippi River.

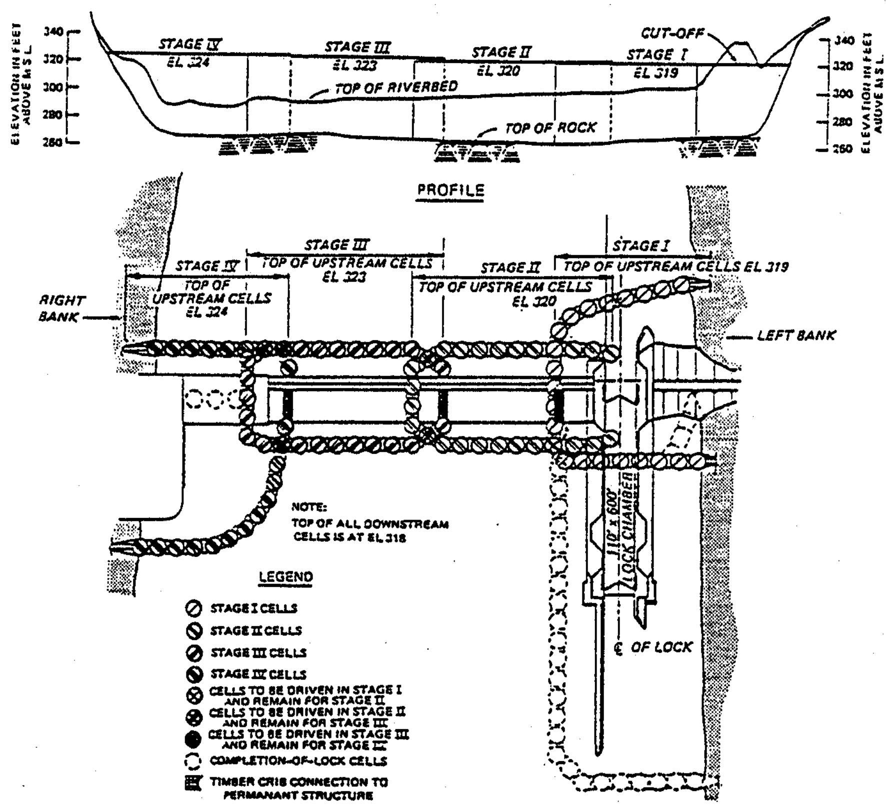


Figure 8.7. Four-stage diversion plan, Dardanelle Lock and Dam, Arkansas River.

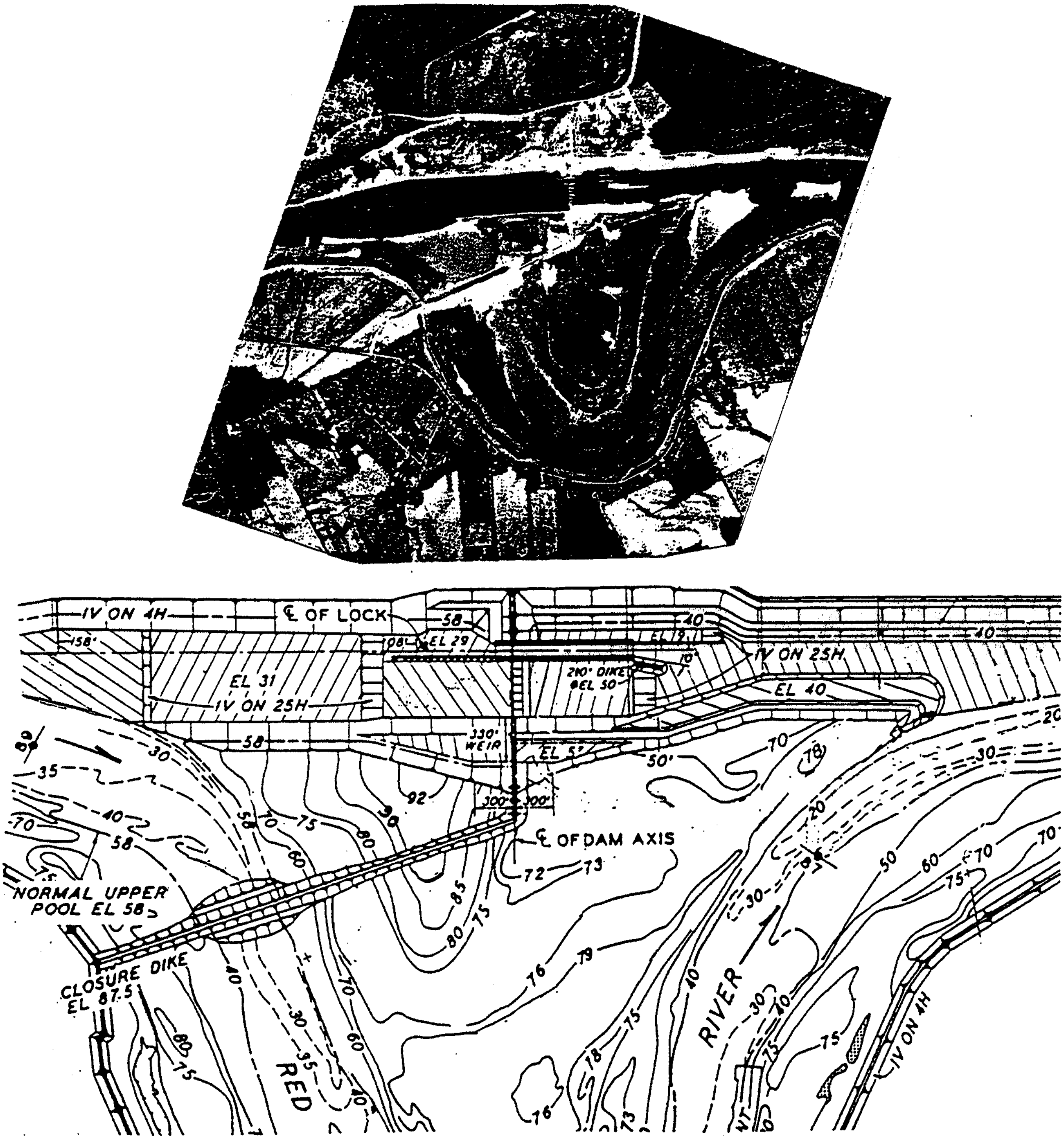


Figure 8.8. Red River Locks and Dams constructed in dry in cutoffs on rectified river alignment.

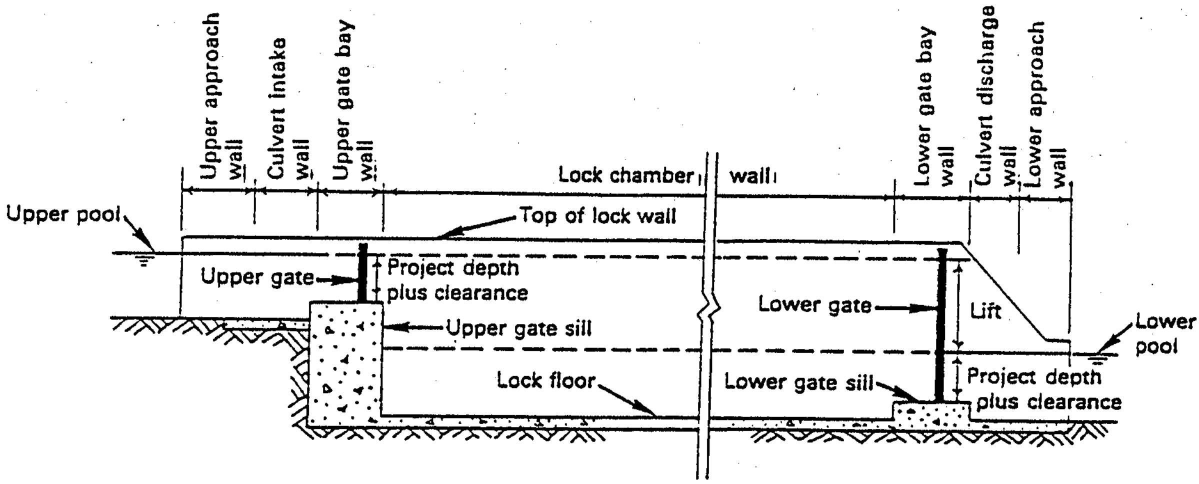


Figure 8.9. Lock walls, gates, and sills.

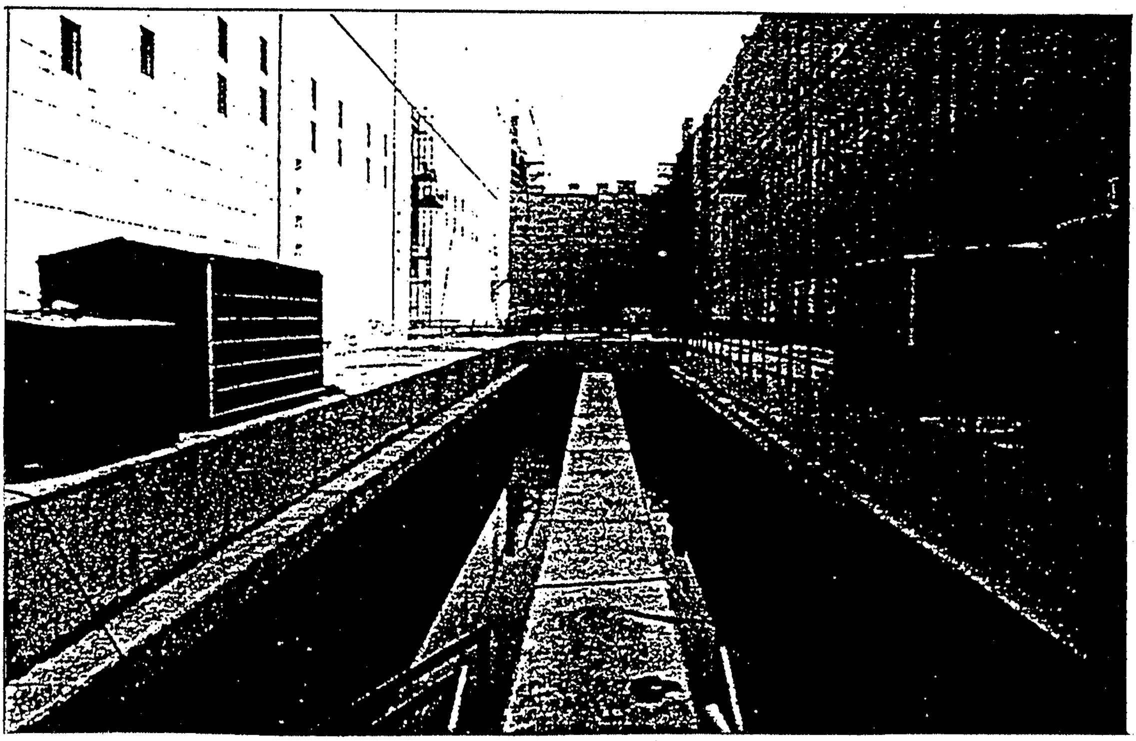
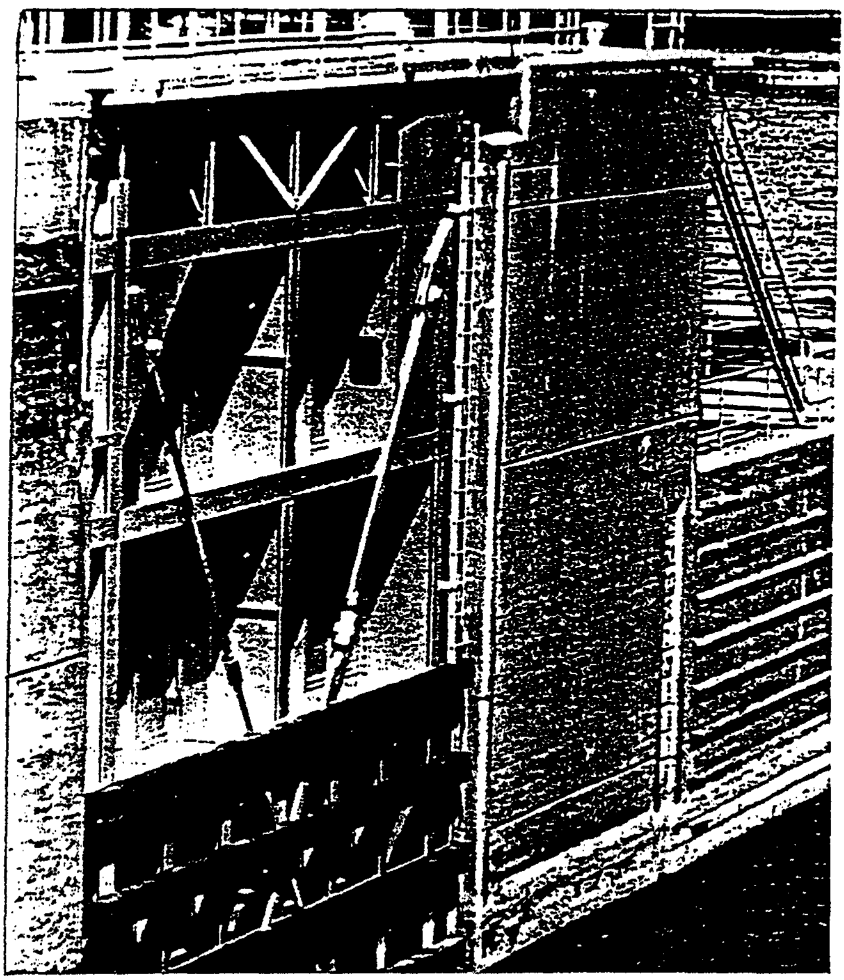


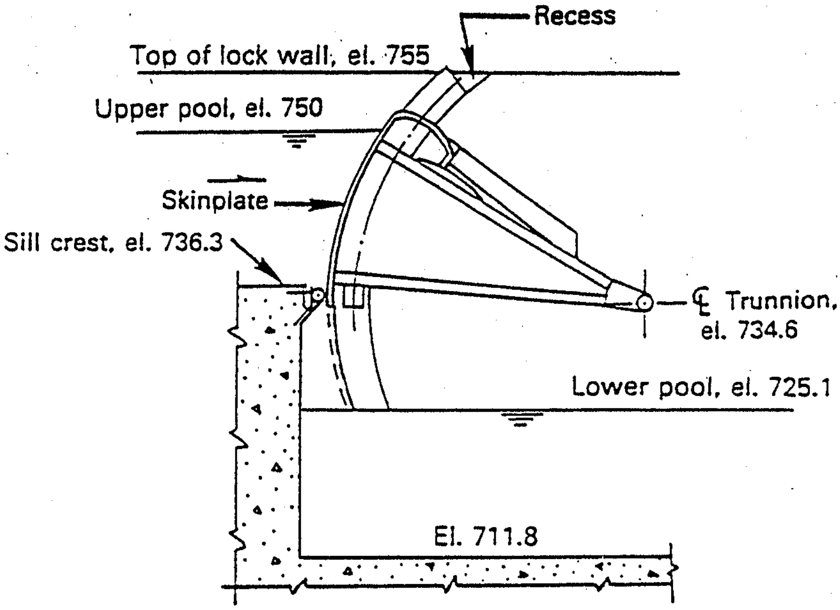
Figure 8.10. Bay Springs Lock chamber during construction, looking at upstream gate sill, Tennessee-Tombigbee Waterway. (U.S. Army, Corps of Engineers, Nashville District.)

(a) Lower lock gate partially open (view from inside lock chamber)



(b) Lower gate fully open and recessed in lock wall

Figure 8.11. Lock miter gates, Lower St. Anthony Falls Lock and Dam, Upper Mississippi River.



(a) Detail (after Nelson and Johnson, 1964)

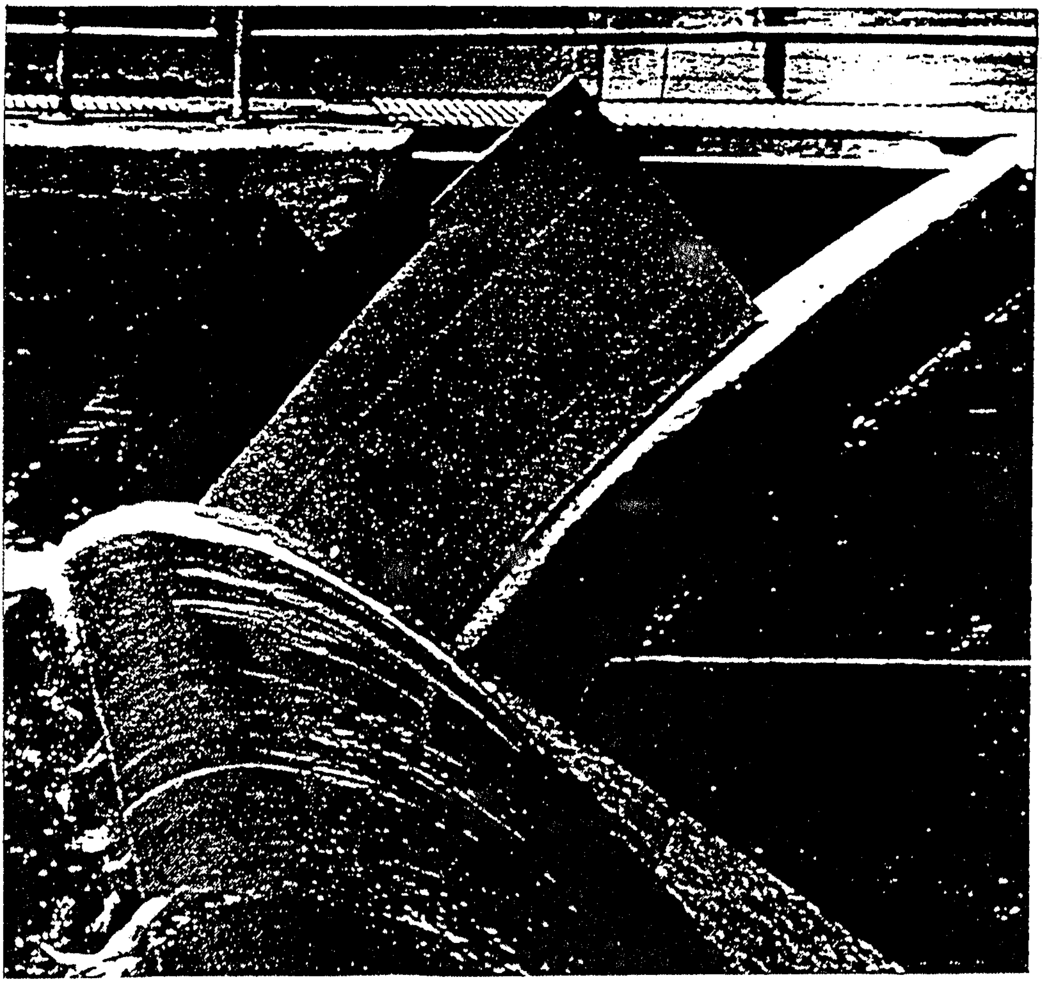
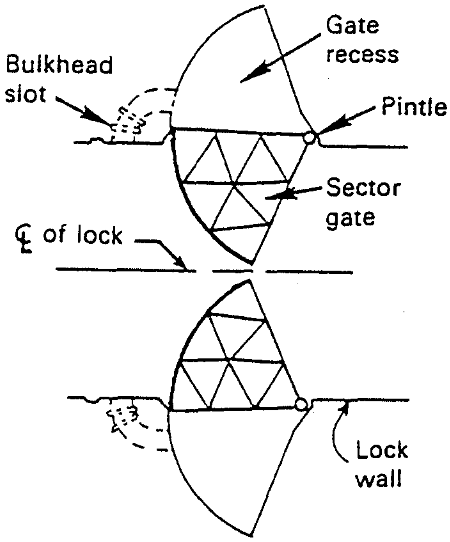
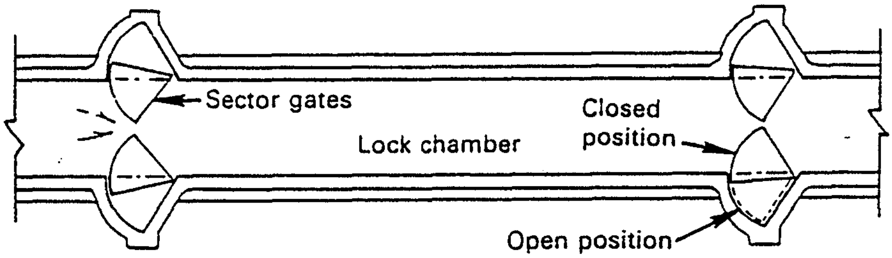


Figure 8.12. Submergible tainter gate (56 ft. long), Lower St. Anthony Falls Lock, Upper Mississippi River.



(b) Sector gates, O'Brien Lock,
Calumet-Sag Project, Illinois
(after Nelson and Johnson, 1964)



(a) Plan

Figure 8.13. Sector gates.

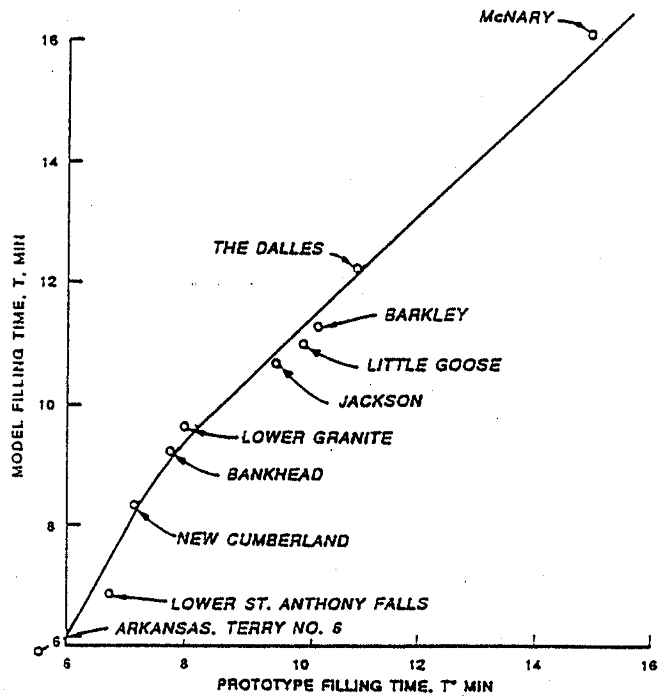


Figure 8.14. Prototype vs model filling time.

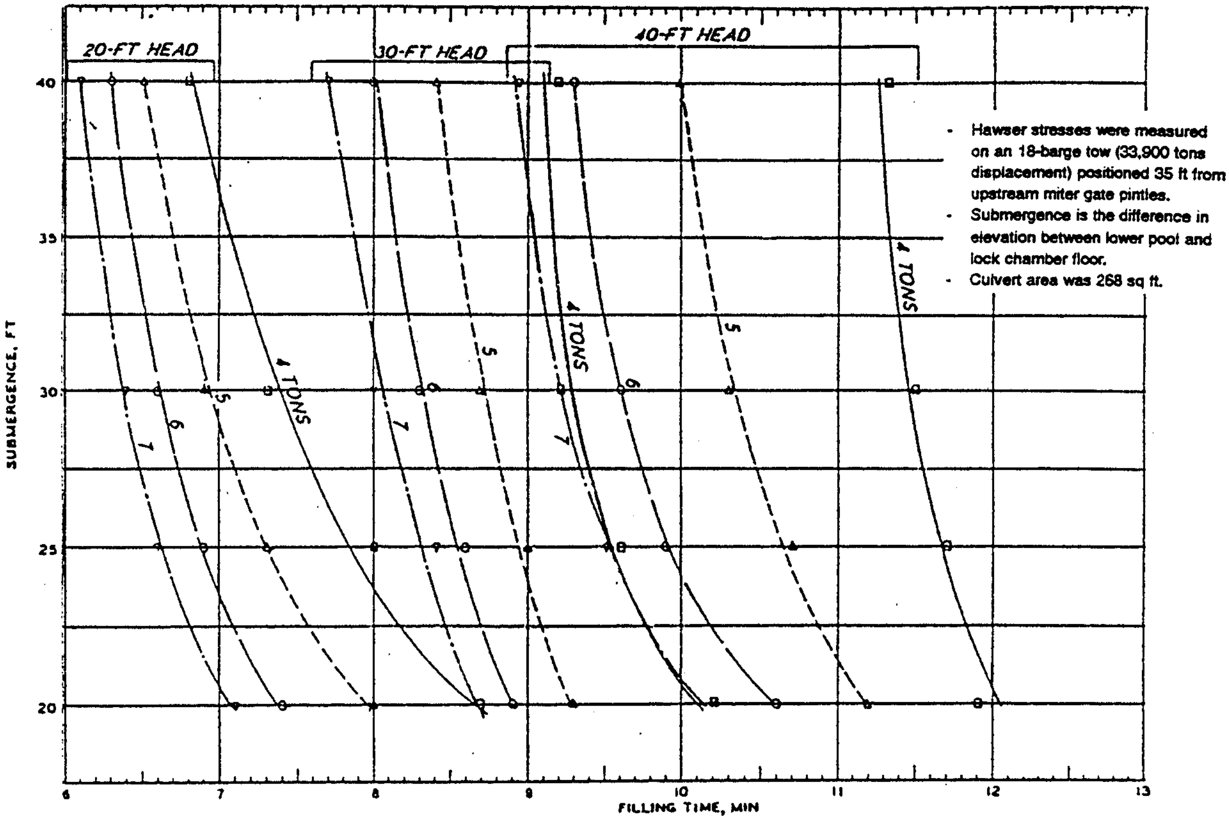


Figure 8.15. Permissible filling time to keep hawser stresses within 4-, 5-, 6-, and 7-ton limits, 110- by 1200-ft lock.

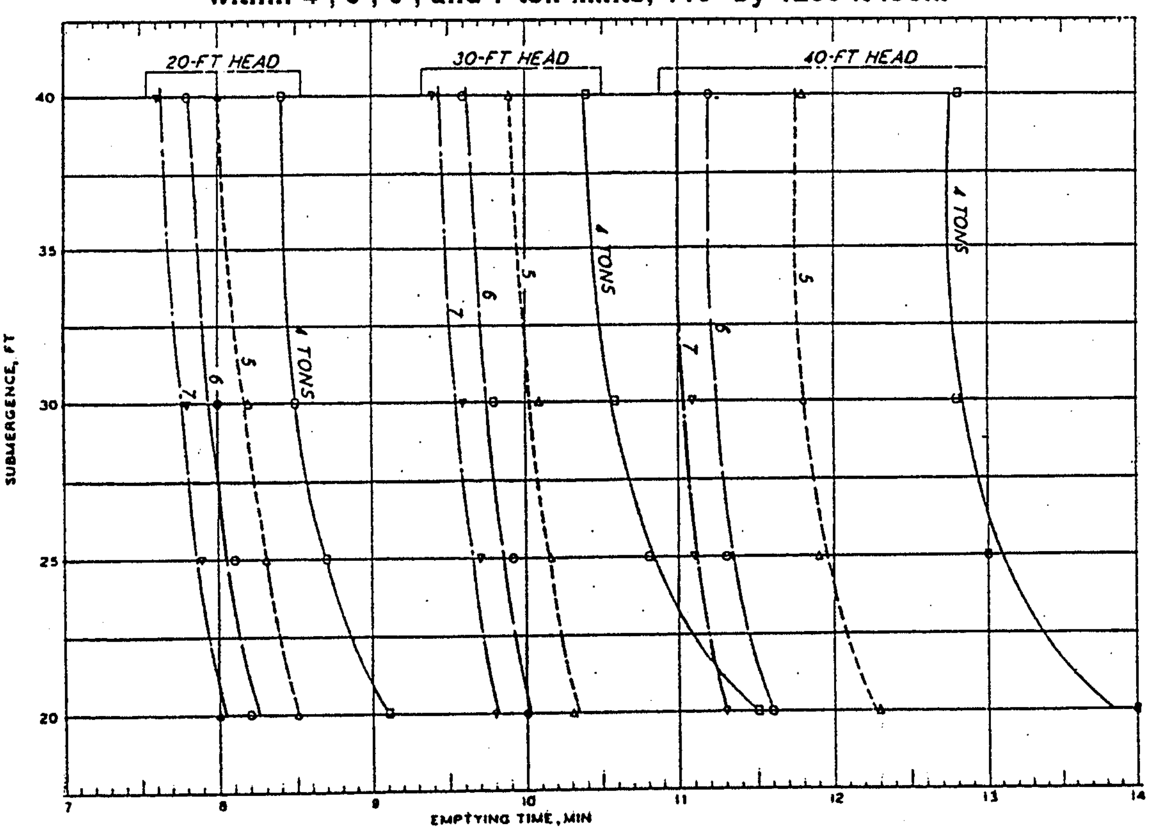
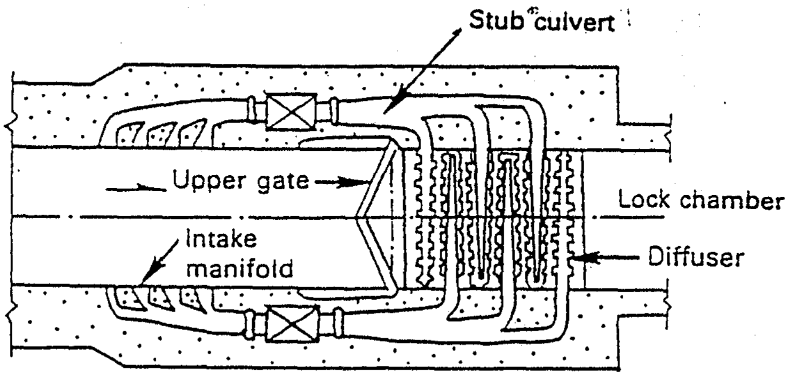
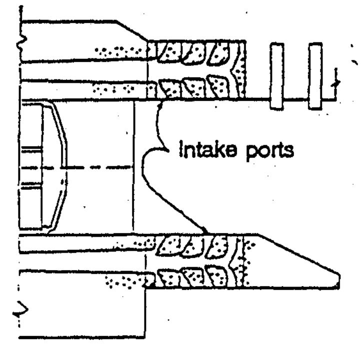


Figure 8.16. Permissible emptying times to keep hawser stresses within 4-, 5-, 6-, and 7-ton limits, 110- by 1200-ft lock.

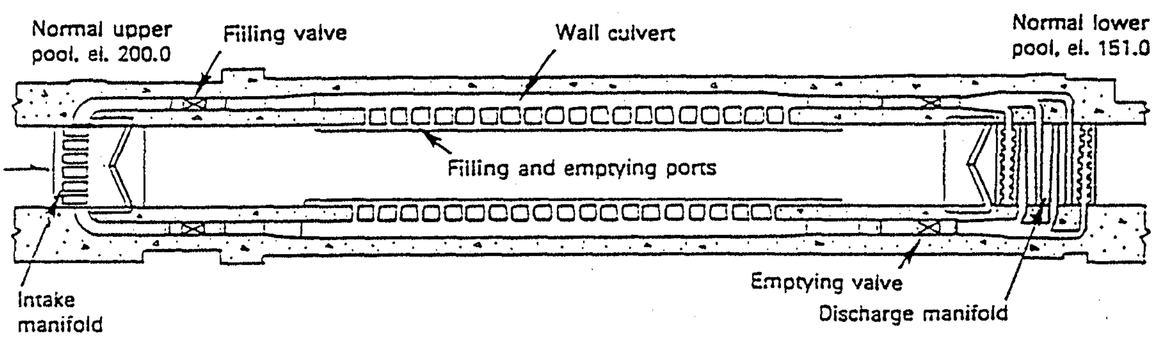


a. **Stub culvert system with diffuser. (Nelson and Johnson, 1964).**

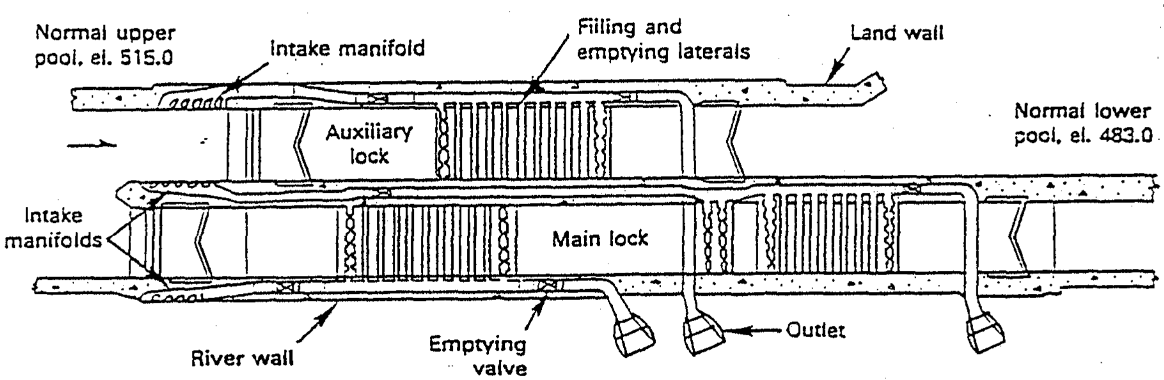


b. **Siamese system, Barkley Lock. (U.S. Army, Corps of Engineers).**

Figure 8.17. Typical lock Filling systems.



a. **Snell Lock, St. Lawrence Seaway (lift 49 ft).**



b. **Greenup Locks, Ohio River (lift 32 ft).**

Figure 8.18. Typical low-lift lock filling and emptying systems. (Nelson and Johnson, 1964).

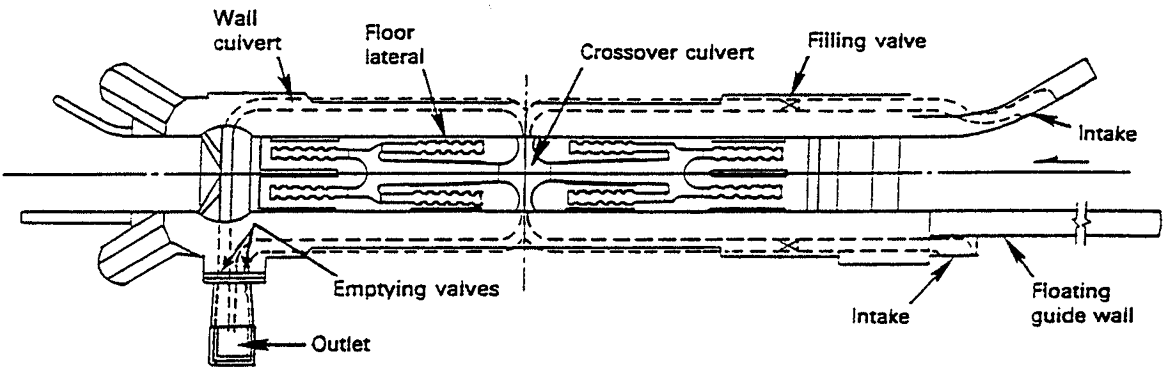
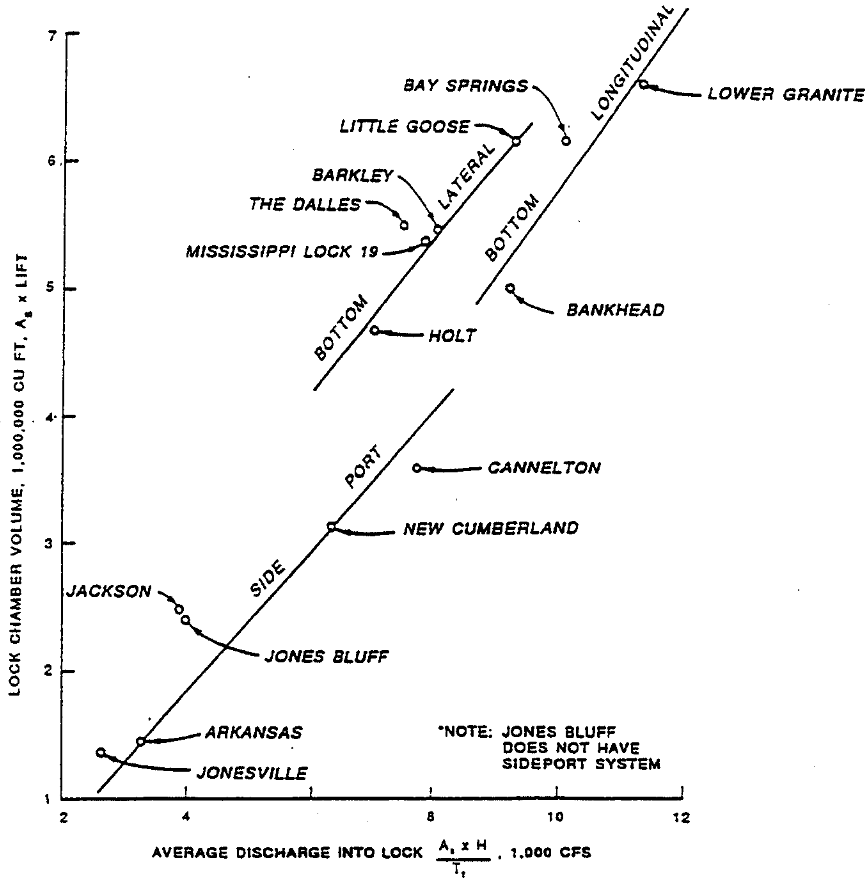
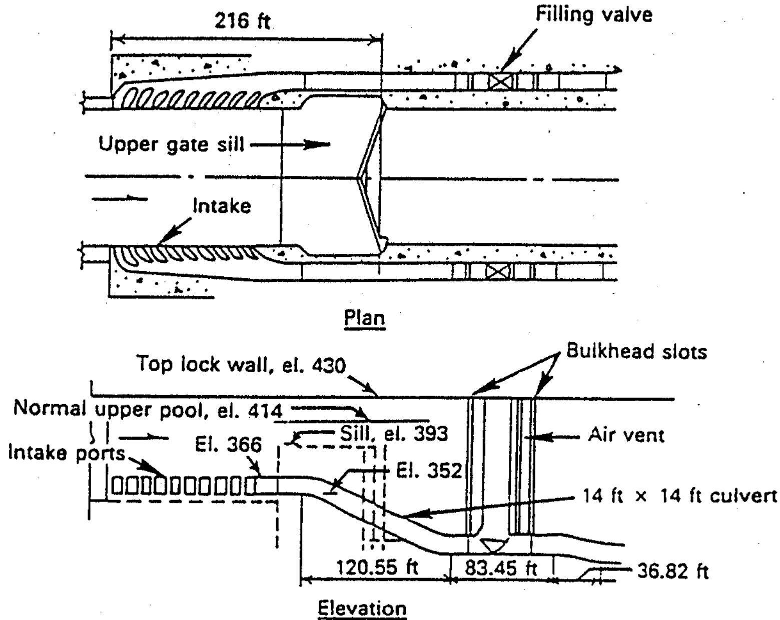


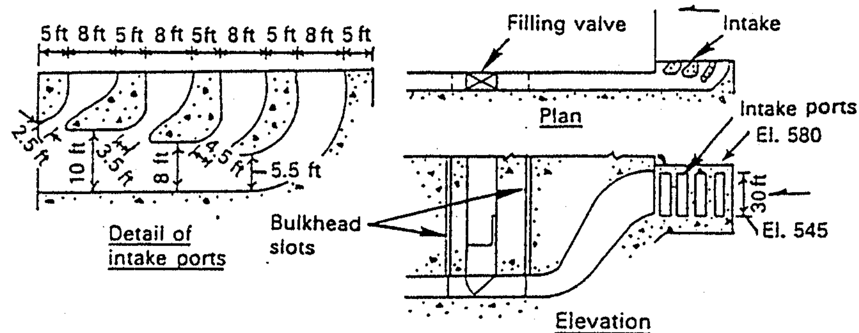
Figure 8.19. Lower Granite Lock, Snake River. (Murphy, 1980).



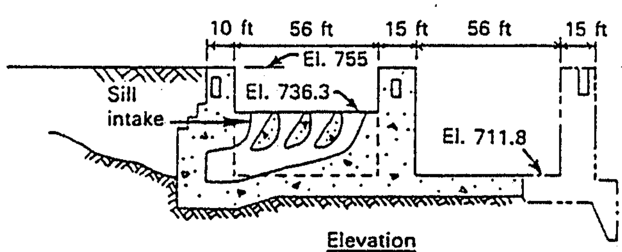
8.20. Lock volume vs average discharge (model filling time). (Davis, 1989).



a. Bay Springs Lock, Tennessee-Tombigbee Waterway.

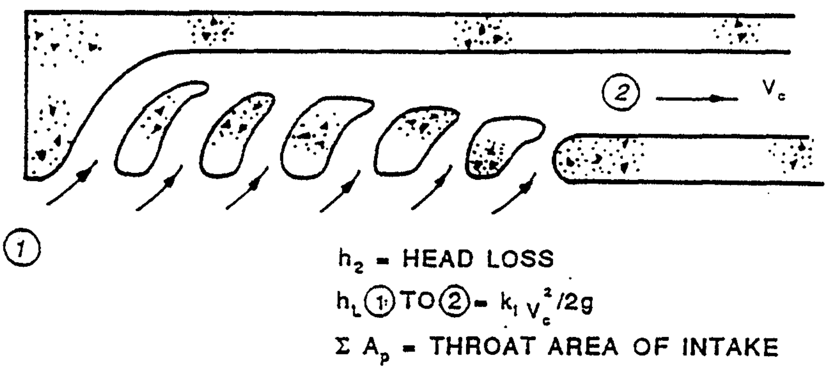


b. Ice Harbor Lock, Snake River.



c. St. Anthony Falls Lower Lock, Upper Mississippi River.

Figure 8.21. Typical intake manifolds (U.S. Army, Corps of Engineers).



Sectional plan.

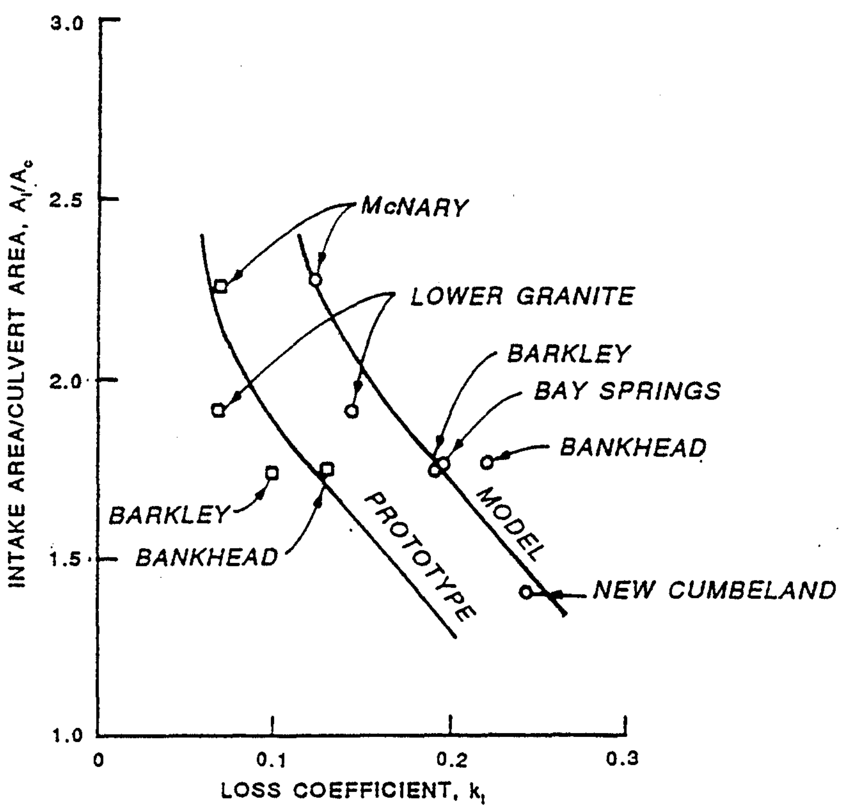


Figure 8.22. Intake head loss coefficient (Davis, 1989).

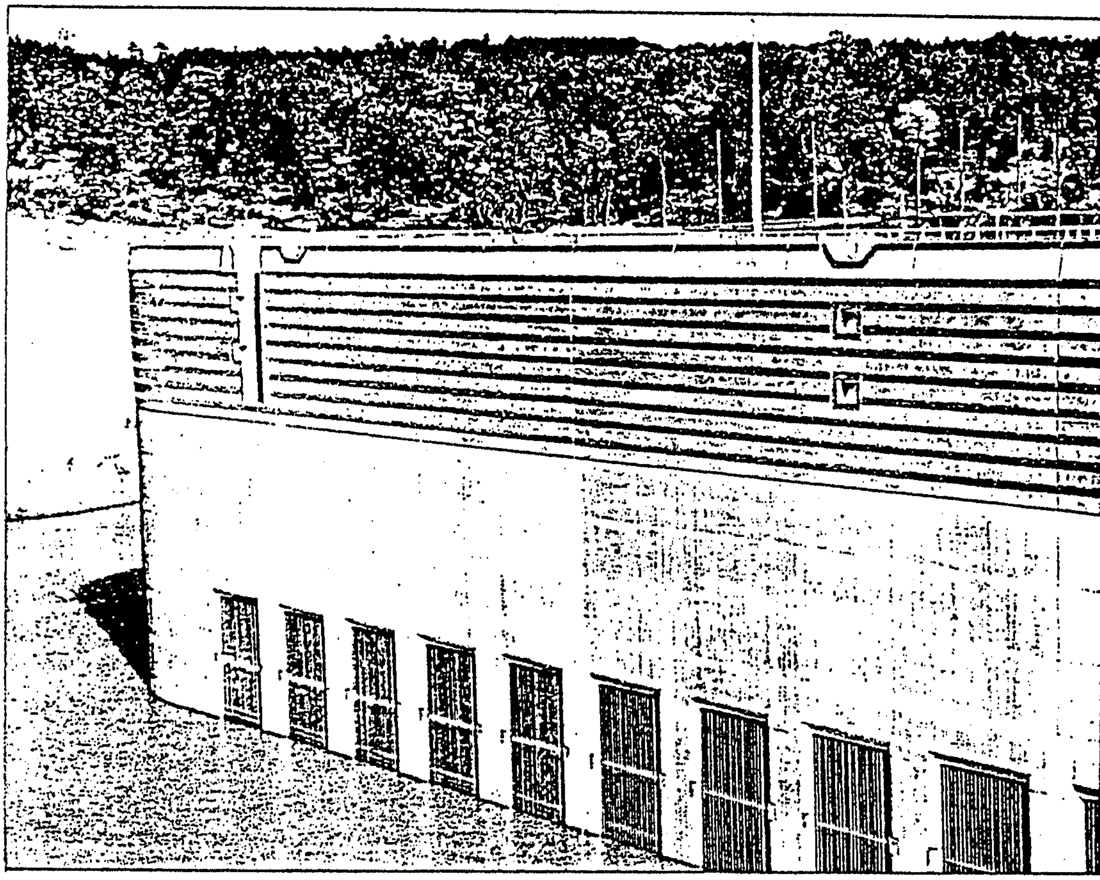
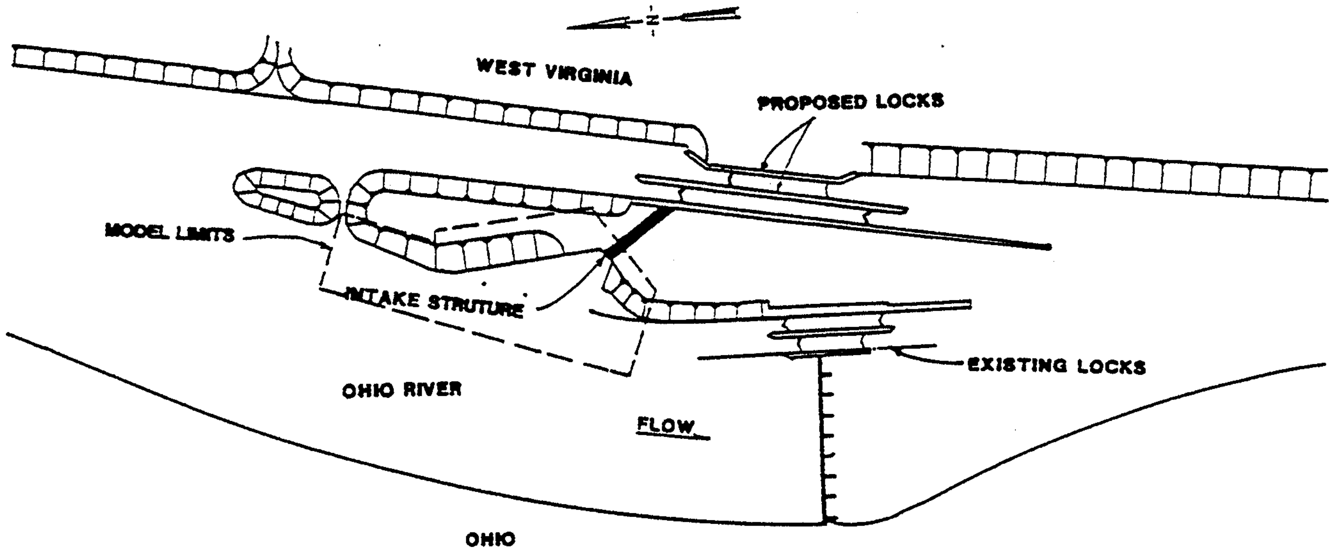
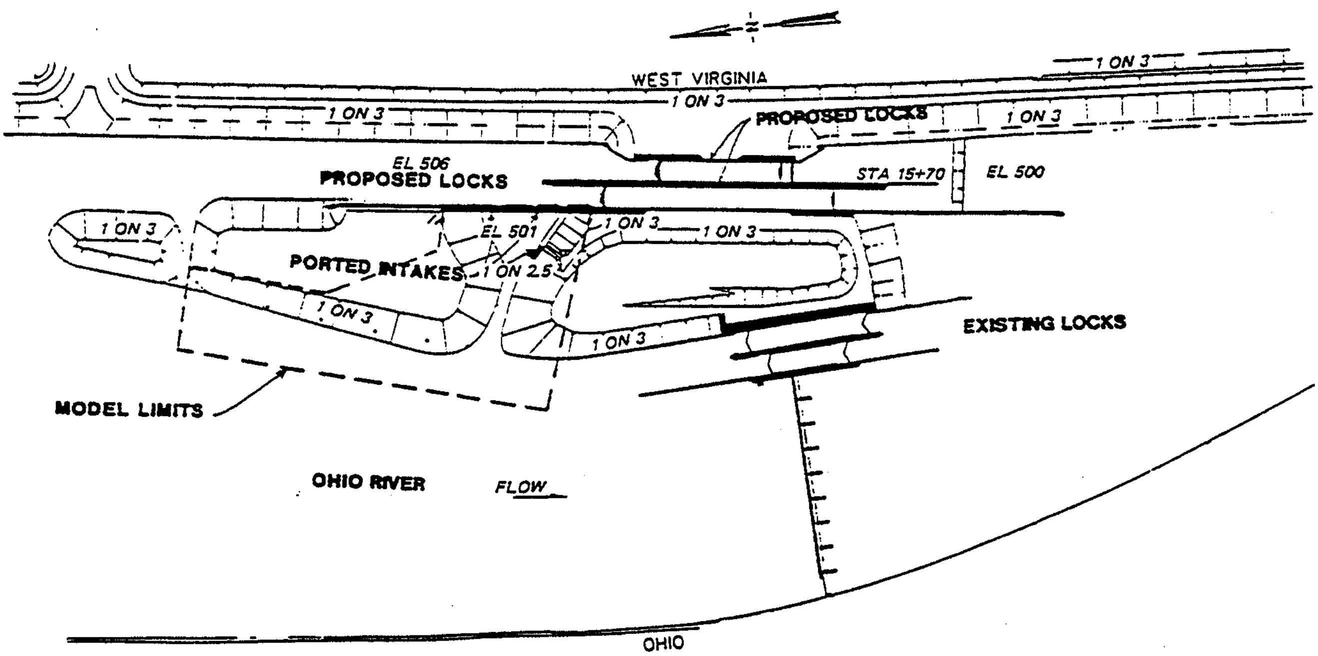


Figure 8.23. Intake ports with trash racks in place during construction, Dardanelle Lock, Arkansas River.

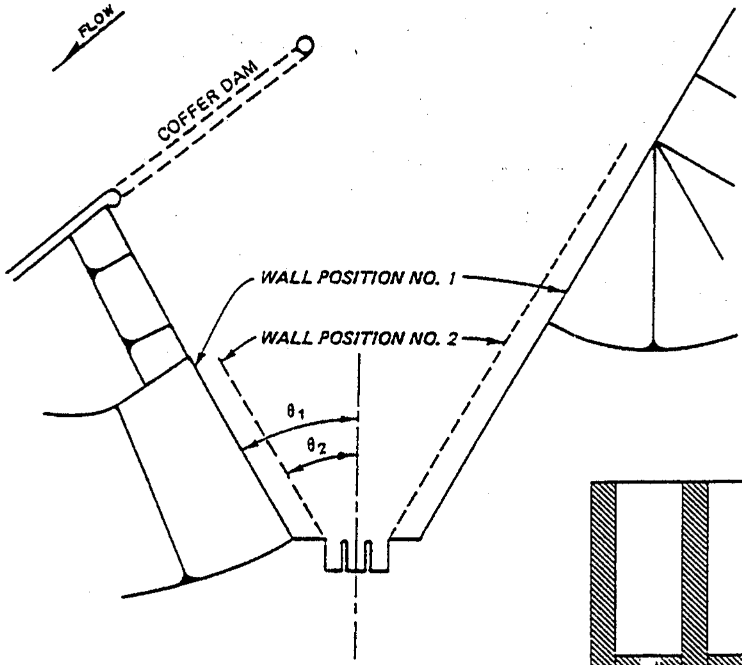


a. Alternative I.

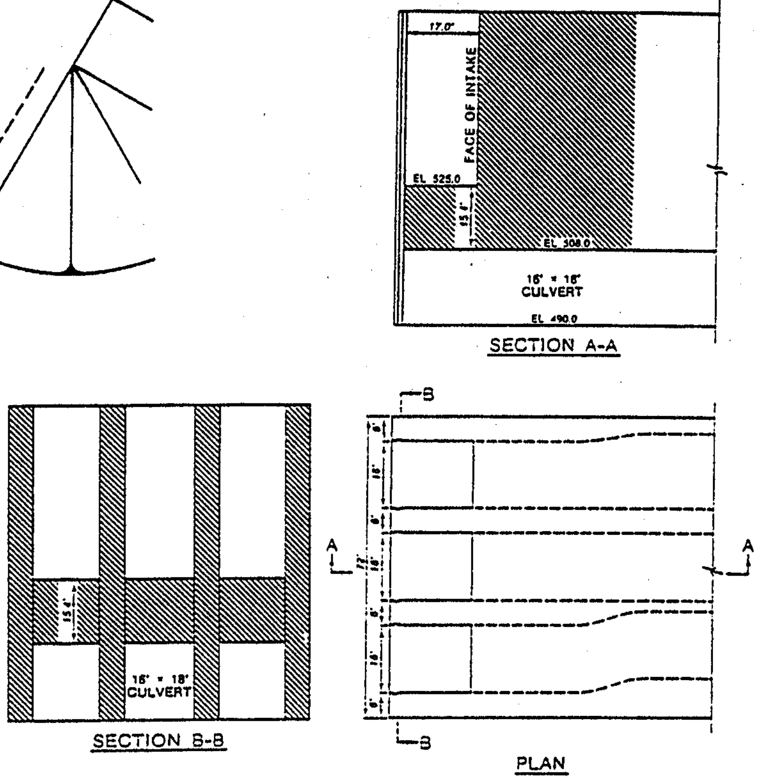


b. Alternative II.

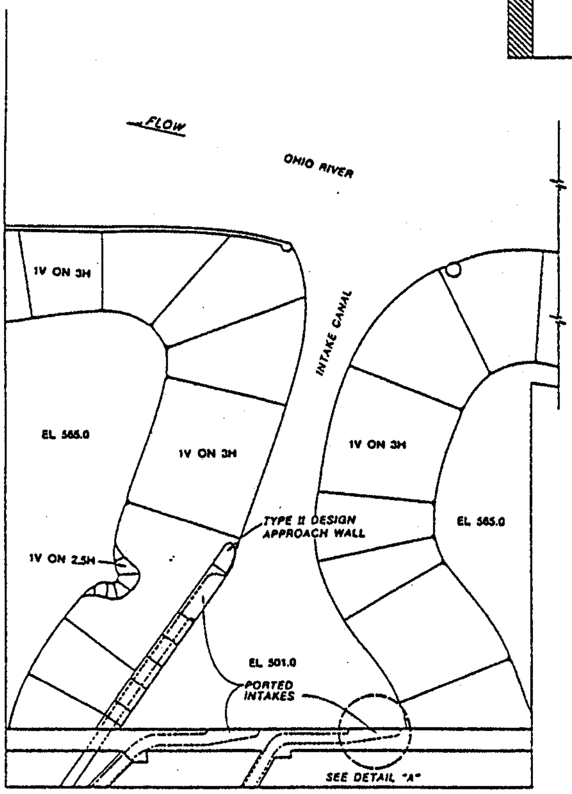
Figure 8.24. Alternative filling schemes, Gallipolis Locks and Dam, Ohio River (Davidson, 1987).

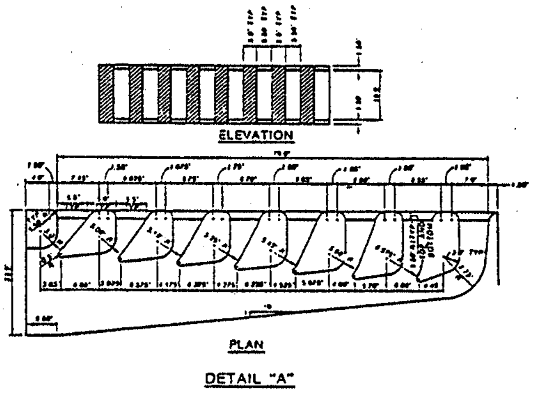


a. Intake approach, Alternative I.



b. Intake structure, Alternative I.





c. Alternative II, Intake canal and manifold.

Figure 8.25. Intakes for alternative filling schemes, Gallipolis Locks and Dam, Ohio River (Davidson, 1987).

V_c = Mean Velocity in Culvert at Valve

V_{vc} = Mean Velocity in Vena Contracta

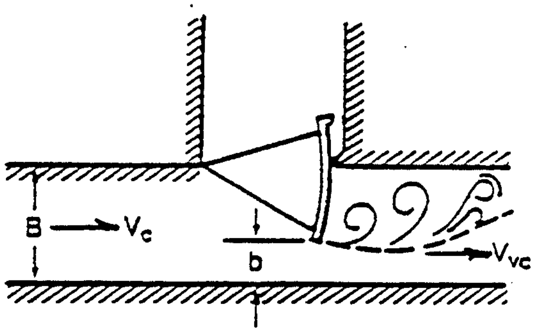


Figure 8.26. Culvert control valve ("reverse" tainter gate).

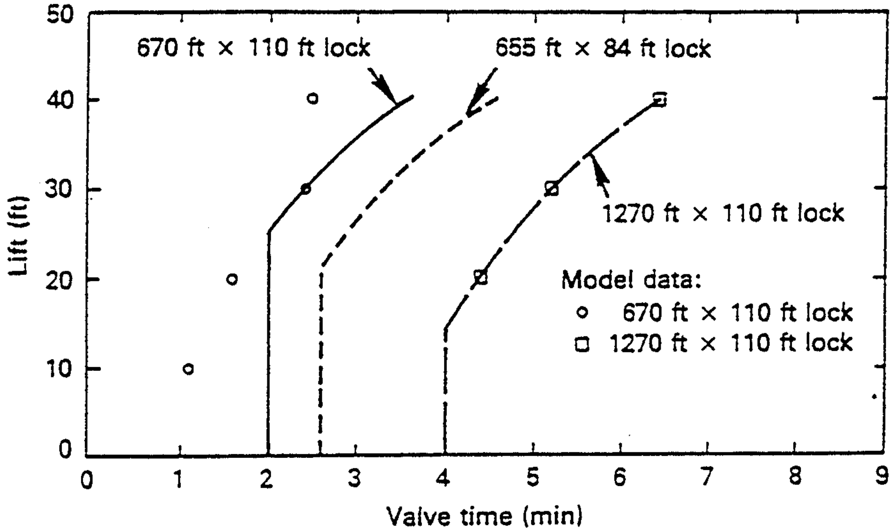


Figure 8.27. Recommended prototype valve opening times for filling (Murphy, 1975).

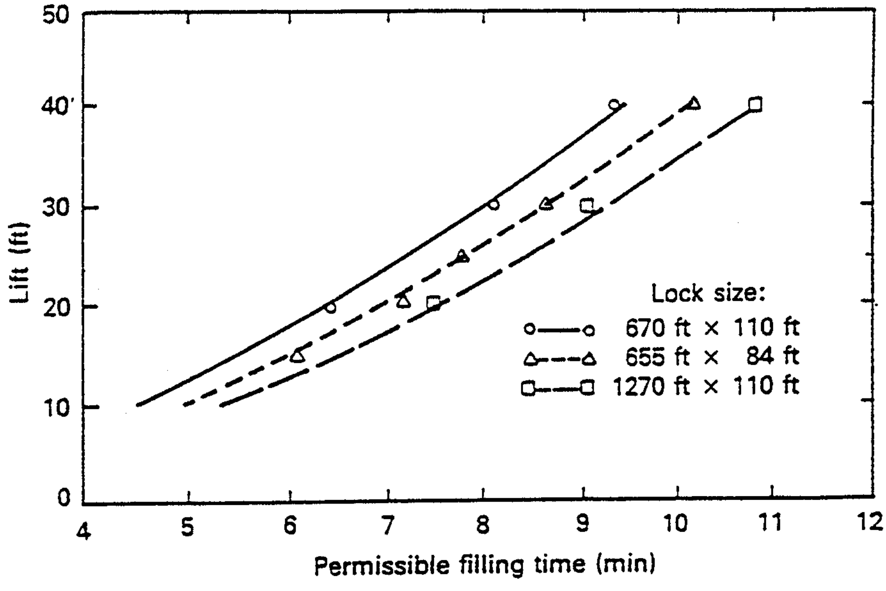


Figure 8.28. Permissible filling times (model) (Murphy, 1975).

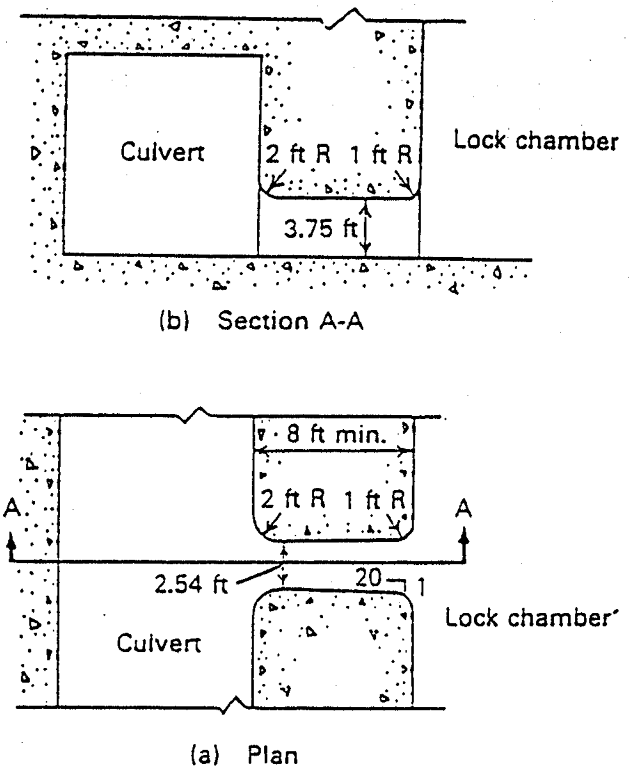


Figure 8.29. Ports for 110-ft-wide lock. (Murphy, 1975).

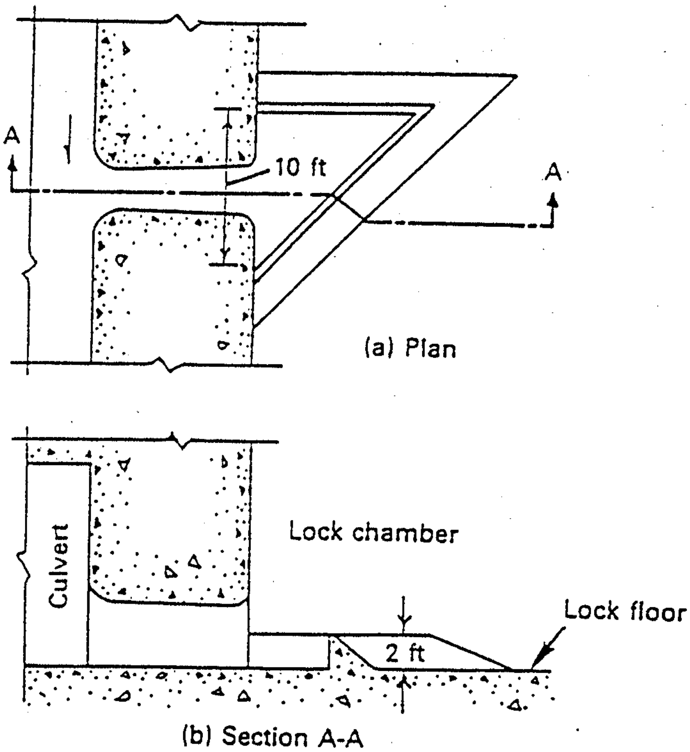


Figure 8.30. Port deflector for 110-ft-wide lock. (Murphy, 1975).

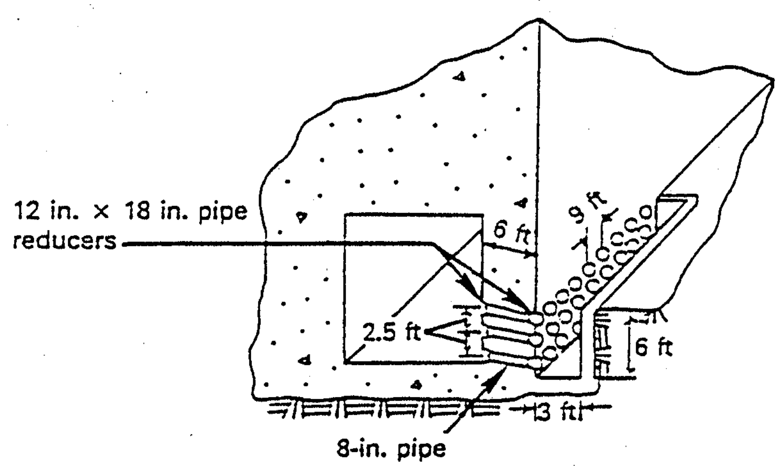
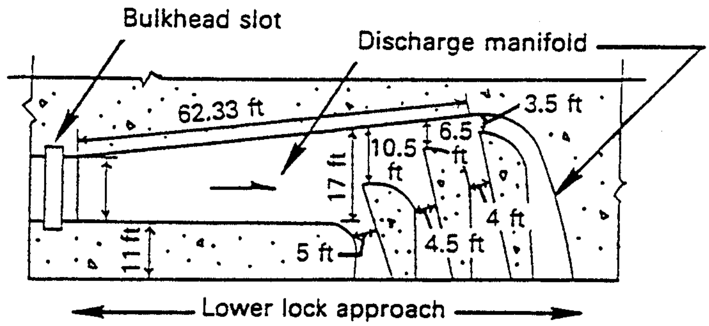
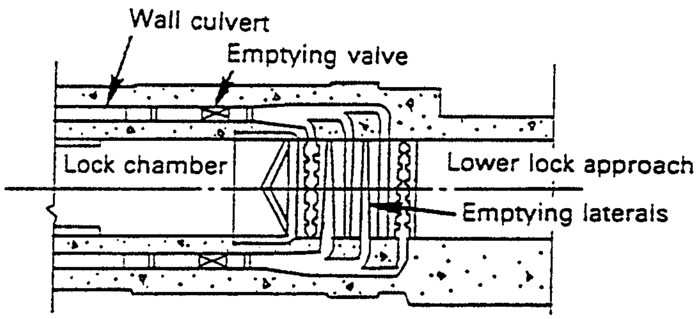


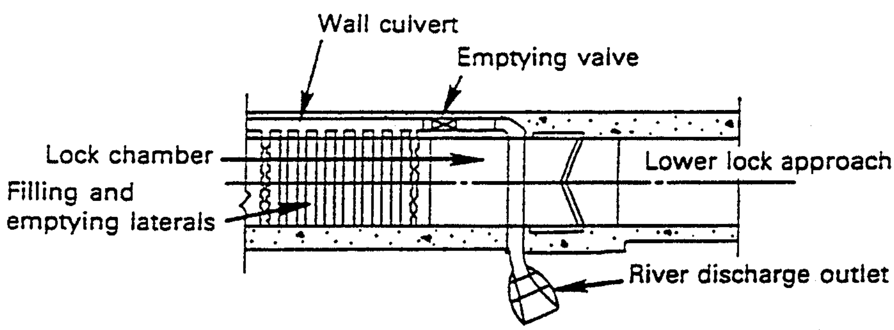
Figure 8.31. TVA multiport system (Elder et al., 1964).



a. McArthur Lock, St. Mary's River (Corps of Engineers, 1956).



b. Snell Lock, St. Lawrence Seaway (Nelson and Johnson, 1964).



c. Greenup Lock, Ohio River (Nelson and Johnson, 1964).

Figure 8.32. Typical lock emptying systems.

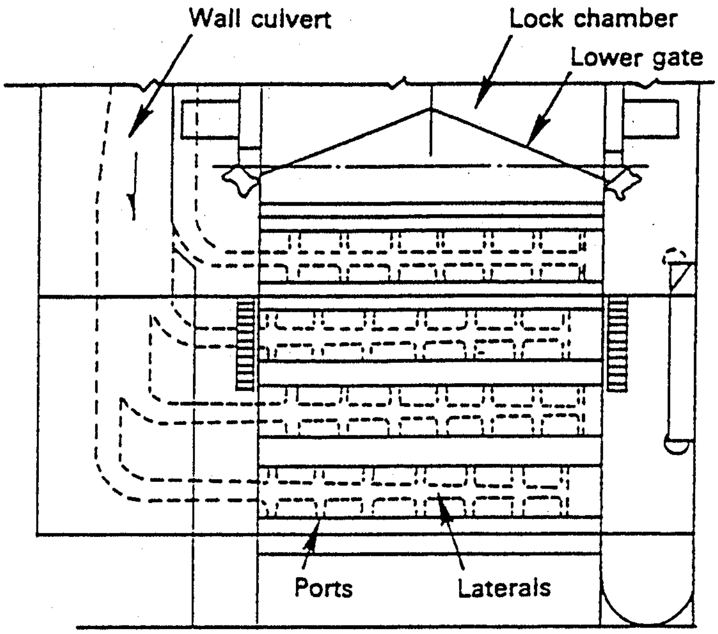


Figure 8.33. Discharge diffuser, St. Anthony Falls Lower Lock, Mississippi River. (U.S. Army, Corps of Engineers, 1956).

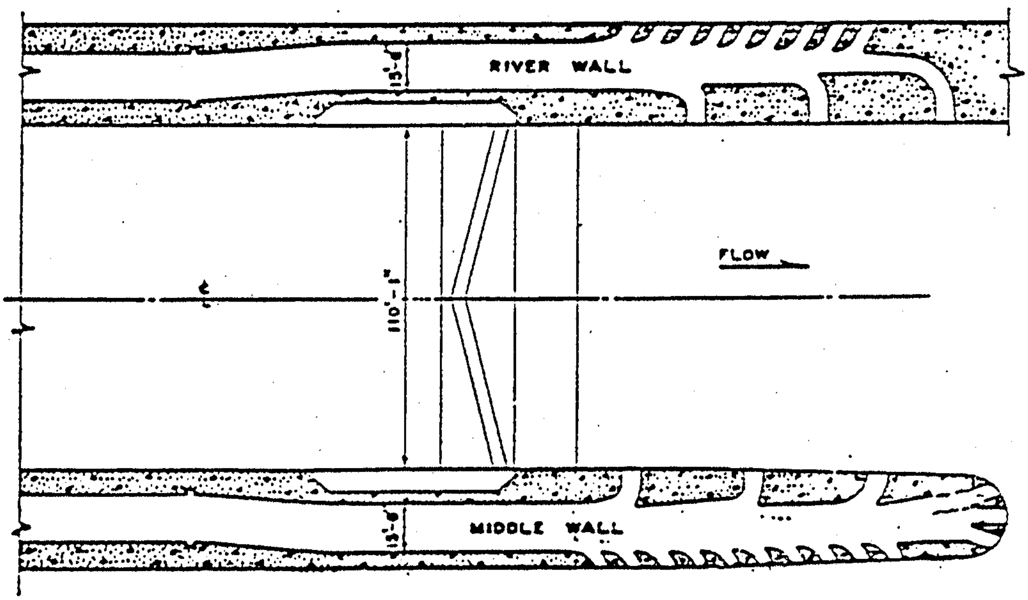
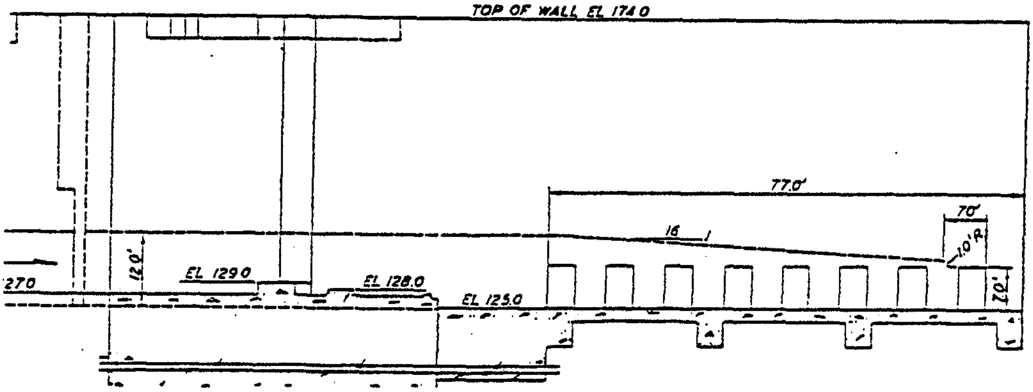
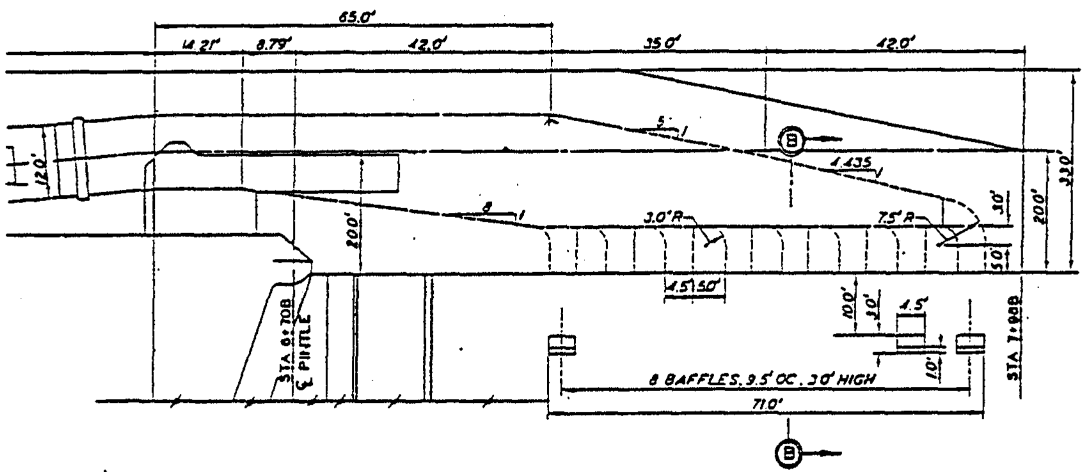


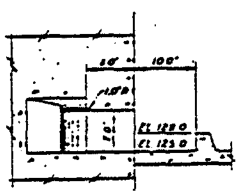
Figure 8.34. Discharge manifolds, New Cumberland Lock, Ohio River (Davis, 1989).



b. Plan

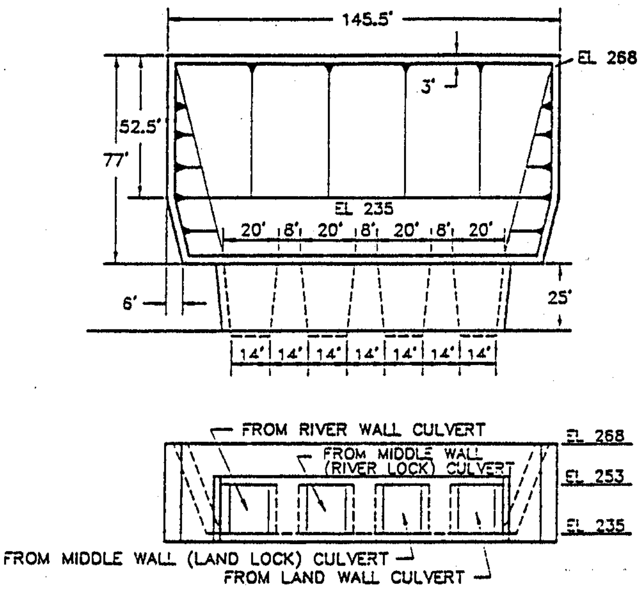


a. Section

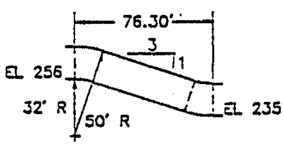


b. Section B-B

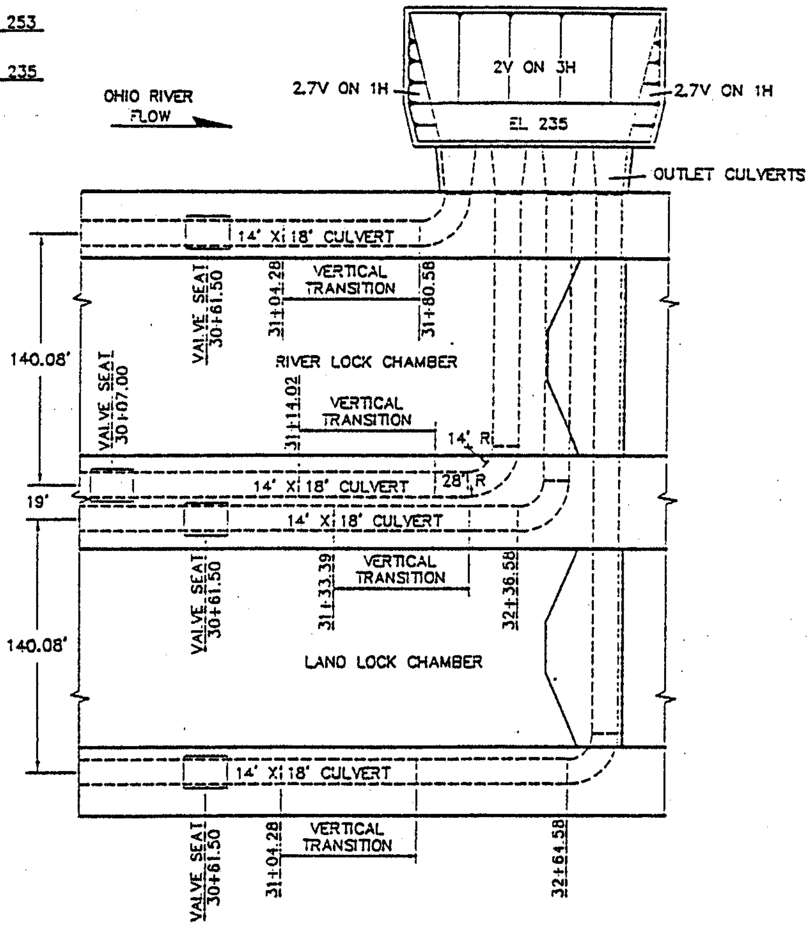
Figure 8.35. Discharge manifold with baffles, Arkansas River (Davis, 1989).



c. Culvert and bucket detail.

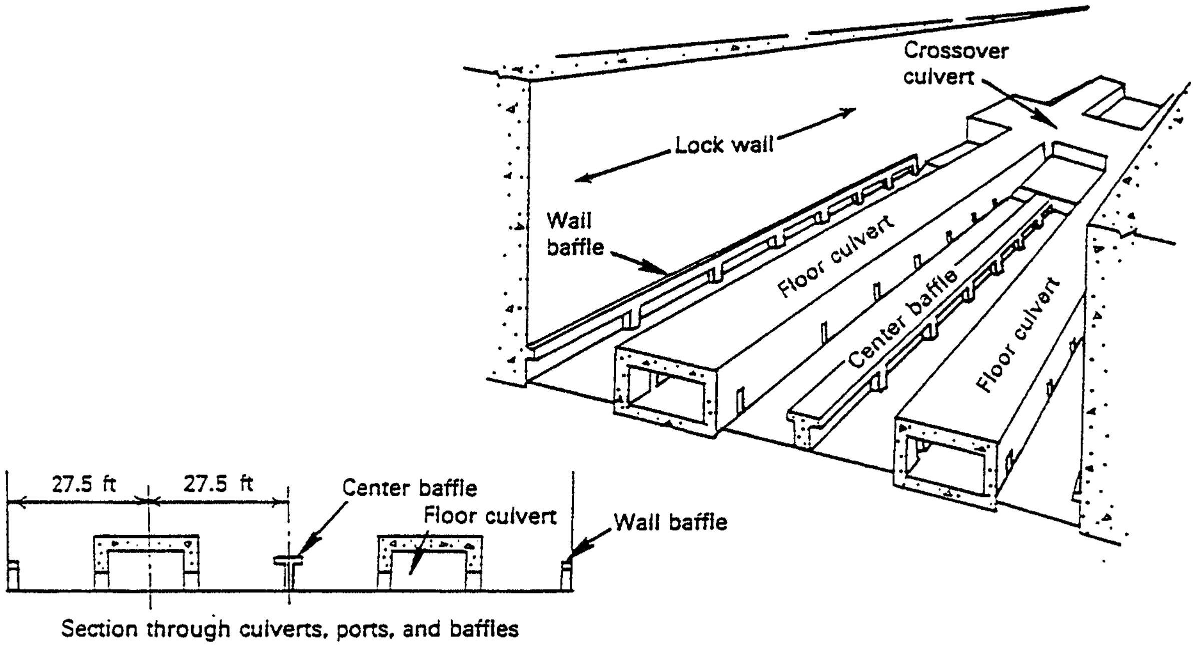


b. Vertical transition.

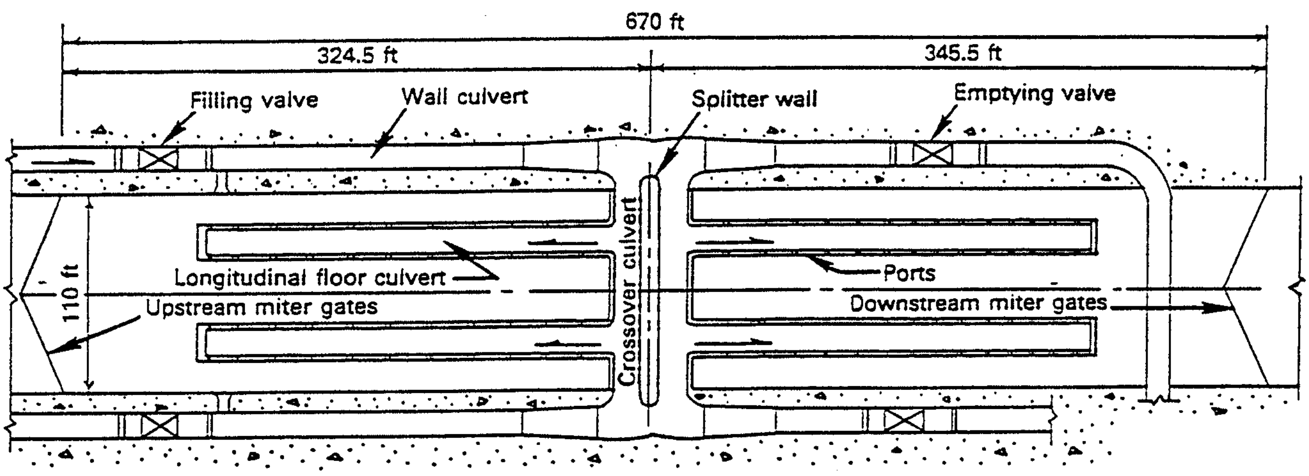


a. Plan.

Figure 8.36. Outlet system, Olmsted Locks, Ohio River (Stockstill, 1992).



b. Culverts, ports, and baffles.



a. Plan.

Figure 8.37. Bottom longitudinal "side-by-side" filling and emptying system, Dardanelle Lock, Arkansas River. (Ables and Boyd, 1969.)

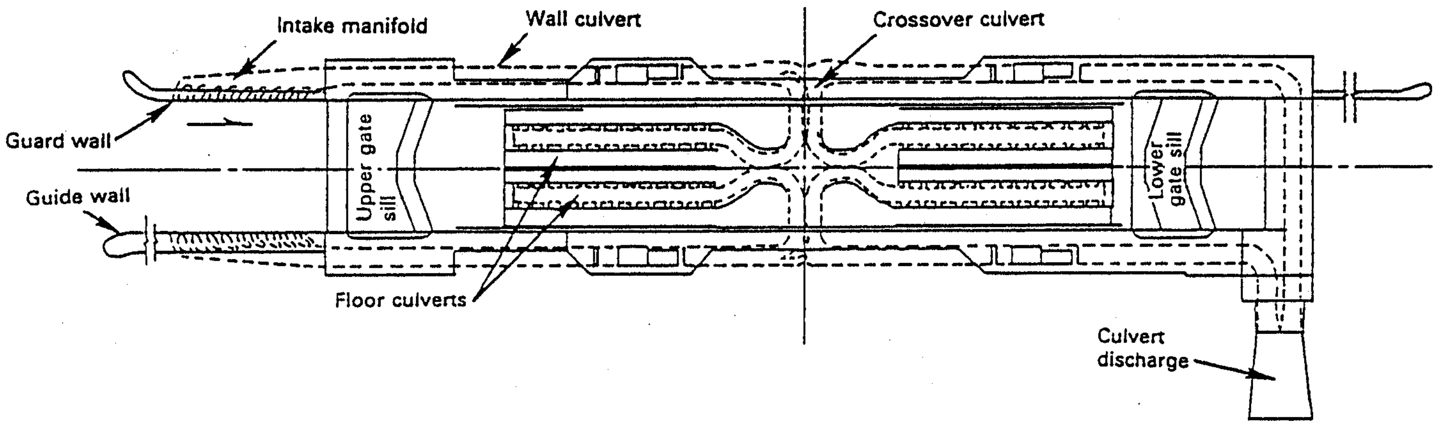


Figure 8.38. Bottom longitudinal "over-and-under" filling and emptying system, Bankhead Lock, 69-ft lift, Black Warrior River, Alabama (Murphy, 1980).

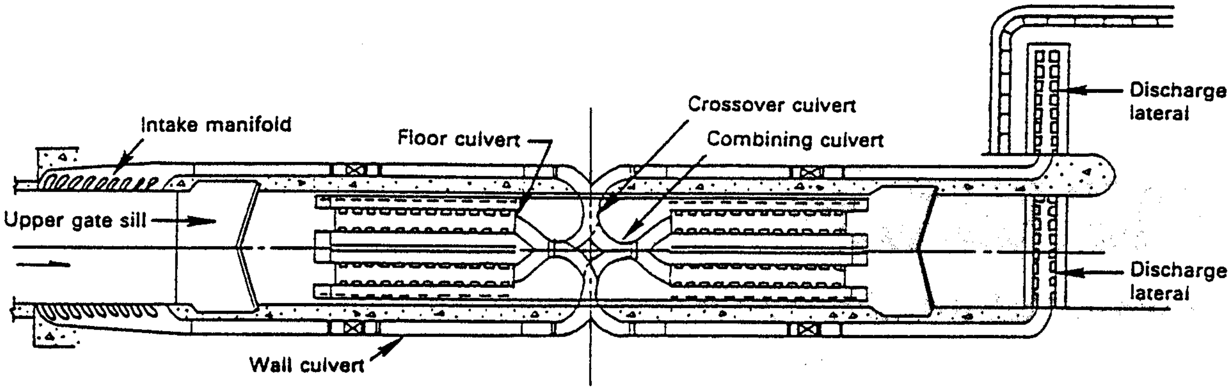


Figure 8.39. Bottom longitudinal "over-and-under" filling and emptying system, Bay Springs Lock, 84-ft lift, Tennessee-Tombigbee Waterway. (Ables, 1978).

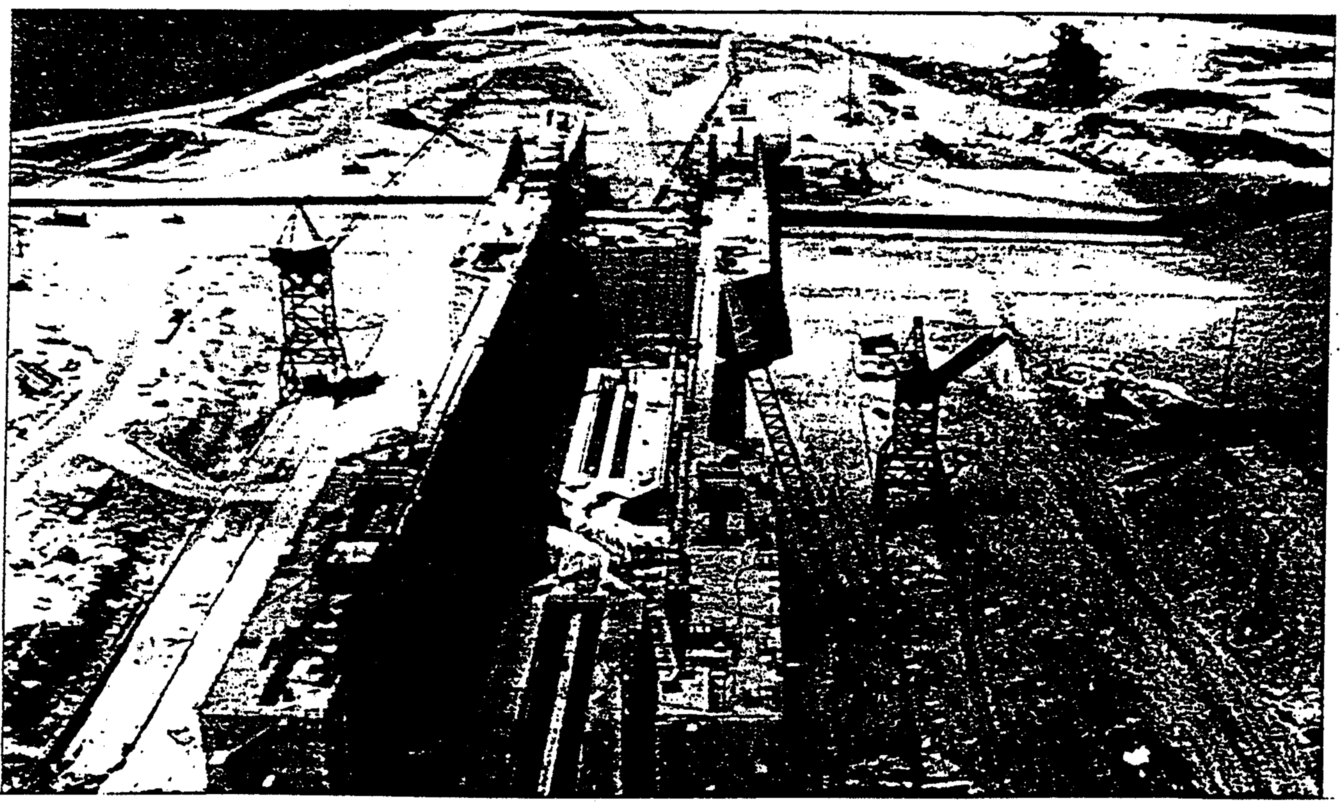


Figure 8.40. Bay Springs Lock under construction, Tennessee-Tombigbee Waterway (U.S. Army, Corps of Engineers, Nashville District).

9. NAVIGATION HAZARDS AT LOCKS

Flow conditions in lock approaches and lock chambers and gate sill elevations may present hazards to navigation traffic. Typical problems at specific projects related to currents and shoaling in lock approaches, surges in lock chambers and approaches related to filling and emptying operations, and tow squat are described in this section.

It should be noted that velocities and currents on some canalized rivers become too high for safe and efficient tow operation during floods. Navigation ceases at about the 10-yr recurrence interval flood on the Arkansas, Red, and Upper Mississippi Rivers. On the Arkansas, this is 250,000 cfs at Van Buren, 335,000 at Dardanelle, 350,000 at Little Rock and downstream. On the Red River, this is 125,000 cfs at Shreveport and 145,000 from Alexandria downstream, when mean channel velocity is in the order of 7 ft per sec and maximum velocities exceed 10 ft per sec. Lock and Dam 26 on the Mississippi River goes out of service at about 720,000 cfs.

Navigation locks are usually located in relatively straight reaches and in or near channel crossings in order to obtain adequate site distances in the upstream and downstream approaches. The best sites are cross sections that are somewhat wider than the average stream cross section because they provide sufficient width to compensate for obstruction of flow by the lock and spillway piers. Cross currents resulting from spillway operation (and power plant discharge if power is included in the project) and currents due to the natural channel configuration are important considerations in site selection.

Constriction of the natural channel by a lock usually results in cross currents in the upstream lock approach as flood flows move across the lock entance toward the spillway. Cross currents tend to develop near the upper end of the guard wall, Figure 9.1 The intensity of cross currents can be reduced by constructing ports in the guard wall, to pass flow intercepted by the guard wall, and by reducing velocities in the approach channel by using dikes to redistribute flow across the channel. Also, turbulence and vortices may occur in the immediate vicinity of the structure due to operation of the filling system.

In the downstream lock approach, undesirable and dangerous currents derive from three principal sources:

a. Spillway and power plant discharges.
b. Expansion eddies immediately downstream of the lock.
c. Currents from the lock emptying system.

Currents and velocities from a lock emptying system in the lower lock approach can be dangerous to tows approaching the lock, especially at medium- to high-lift locks. At such locks it may be desirable to locate the discharge manifold outside the lower lock approach, as at Greenup Lock, Figure 8.18b, and at Olmsted, discussed in Section 9.4.

Where releases from the dam expand downstream of a lock, sediment tends to move toward and into the lower lock approach, and the resulting deposition can be a significant problem.

9.1 Dardanelle Lock and Dam, Arkansas River

Model data for two alternative layouts of the power plant at Dardanelle Lock and Dam (lift 54 ft) illustrate the occurrence of cross currents in the upper lock approach with a ported upper guard wall, Figure 9.1. Ports in the upper guard wall reduce cross currents by permitting the flow intercepted by the lock to pass through the wall to the spillway. The effectiveness of ports in reducing cross currents depends on the number, size, and hydraulic efficiency of the ports. Franco (1976) suggested that, in general, the total cross-sectional area of port openings in the upper guard wall should be equivalent to the cross sectional area of the approach channel affected by the lock and lock walls and that the top of guard wall ports should be 4 to 6 ft below the bottom of a loaded tow to minimize pull of the tow toward the wall. Franco also recommended the channel bottom between the guard wall and bank be near or lower than the bottom of the ports to reduce velocities and prevent build up of head on the landside of the tow. When ports extend down to the stream bed, an alluvial bed should be protected against scour. Velocities in the upper approach in the Dardanelle model appeared to be low enough (1 to 2 ft/sec) and sufficiently well aligned with the lock so as to not interfere with tow movement.

Typical patterns and velocities in the lower Dardanelle approach are shown in Figure 9.2, based on model studies (U.S. Army, Corps of Engineers, 1960). Model studies indicated that ports in the lower guard wall were not effective in reducing eddy action downstream of the end of the wall. Based on model studies, all Arkansas River locks have ported upstream guard walls and solid downstream guard walls.

9.2 Lock and Dam 2, Red River

Lock and Dam 2 on the Red River (lift 24 ft) was completed in 1987. After completion, navigation conditions in the upstream lock approach were difficult at medium to high river flows when mean channel velocity was about 7 ft/sec and maximum velocities were in the order of 10 ft/sec. One of the alternative guide wall designs model tested is shown on Figure 9.3a. The upstream guide wall constructed is a 700-ft cellular structure with ports 35-ft wide (except for the most downstream port). When the project went into operation, flows were concentrated through the most downstream ports, and at some discharges, velocities were sufficiently high through the ports to pin tows against the wall. It was concluded that:

a. Flows through the ports should be redistributed to be more uniform.

b. Lateral flow distribution in the upstream reach of river should be altered to reduce the percent of total river flow entering the lock approach.

Robertson (1995) reported the following remedial measures were taken. A system of submerged dikes was installed upstream from the guide wall to force flow away from the lock side of the river; top of dikes was 14 ft below normal pool level. Flow conditions in the upstream approach improved immediately, and much less debris collected in front of the upper miter gates. In the next high-water season, river pilots reported it was much easier to enter and leave the lock with the dikes in place. The effect of such dikes is shown schematically in Figure 9.3b. Similar submerged dikes were installed initially at several Arkansas River locks having similar approach problems.

Unequal distribution of flow through ports in the guide wall was still a problem, however. Prototype measurements indicated that 60 percent of the flow entering that portion of river bounded by the guidewall passed through the downstream 25 percent of the wall. To redistribute the flow, concrete blocks were placed in the three full-sized ports at the downstream end of the wall, reducing flow through those ports about 50 percent. Approximately 38 percent of the entering flow now passes through the downstream 25 percent of the wall, and the current problem has been solved.

The Red River Waterway is discussed further in Appendix B.

9.3 Robert S. Kerr Lock and Dam, Arkansas River.

The Robert S. Kerr Lock has a 110- by 600-ft lock chamber on the left bank with a maximum lift of 48 ft and a four-unit powerhouse on the right bank. Embankments above maximum pool level connect the lock and dam to high ground on both banks. The ogee spillway has 18 tainter gates, each 44 ft high and 50 ft wide. General reach conditions prior to construction of the project and limits of the model are shown on Figure 9.4a. The structure layout and details are shown on Figure 9.4b; it will be noted that the 600-ft upper guide wall has 25-ft diameter sheet pile cells on 50-ft centers. Thus, the ports in the upper guide wall are 25 ft wide and 37 ft high.

Navigation conditions in the lock approaches were studied in a 1:120 fixed bed model. Model tests indicated (Franco and Glover, 1968) that with the original design:

a. Downbound tows approaching the lock would have difficulty because of high cross currents near the end of the upper guide wall caused by flow from the left overbank moving across the upper lock approach to the spillway, Figure 9.5.

b. Upbound tows approaching the lower guard wall would experience considerable difficulty in the lower approach due to the strong eddy that formed with the powerhouse in operation and no flow through the spillway. Velocities as high as 2.9 ft/sec cut across the navigation channel near the end of the lower guidewall, Figure 9.5. No problems should be encountred in the lower approach with the spillway in operation, Figure 9.5.

c. Tows passing under the bridge downstream of the lock would experience some difficulty.

Modifications (Plan C) in the model indicated that safe navigation conditions could be obtained in the upper approach by extending a fill from the left dam embankment at least 3000 ft upstream (top of the fill would be above the flow line for the maximum navigable discharge), Figure 9.6. The fill along the left side of the upstream lock approach forces cross currents from flow from the left overbank to move across the approach channel farther upstream where downbound tows can maintain sufficient speed and approach the upper guide wall without difficulty.

Eliminating ports in the upper guide wall (Plan C-1) increased size and intensity of the eddy along the riverside of the fill, Figure 9.6b. There was a tendency for tows to be moved away from the wall, making it difficult for them to align to enter the lock. With ports in the upper guide wall, tows had less difficulty in aligning for entrance than with a solid wall, but the capacity of the ports could be reduced significantly from that of the original design.

The adverse effects of the strong eddy in the lower lock approach with the powerhouse operating and no flow through the spillway was reduced by modifying the right bank downstream of the powerhouse and extending the lower guard wall with a 550-ft long rock dike. Currents and velocitiies with the powerplant operating with and without spillway discharge are shown in Figure 9.7. Extending the guide wall reduced the intensity of the eddy in the lower approach with the powerhouse in operation and gave tows entering and leaving the lock additional maneuvering area. The small eddy between the guide wall and the left bank did not appear to be of sufficient intensity to affect navigation.

9.4 Olmsted Locks and Dam, Ohio River

Olmsted Locks and Dam (lift 21 ft) is located on the lower Ohio River, 16.6 miles above the confluence of the Ohio and Mississippi Rivers. Tailwater at Olmsted Locks is not affected by a downstream navigation structure; open-river conditions prevail downstream. Tailwater elevation ranges widely and is influenced by Mississippi River backwater levels. There are two 110- by 1200-ft locks with a 21-ft lift, Figure 7.4. The emptying system consists of four wall culverts from the two locks (located in the land wall, middle wall, and river wall) emptying into a single outlet structure in the river, Figure 8.36.

A 1:25 scale model of the outlet for Olmsted Locks was used to investigate flow patterns, velocities and water levels in the vicinity of the outlet structure (Stockstill, 1992). The model reproduced the lock emptying system downstream of the emptying valves, approximately 1150 ft of the Ohio River, beginning 650 ft upstream of the outlet, and approximately 50 ft of the width of the river. Three steady state flow conditions were tested: land lock emptying; river lock emptying; and both locks emptying simultaneously. Unit river discharge was 57 cfs/ft, and the maximum outlet discharge was 10,500 cfs/lock (21,000 cfs with both locks emptying simultaneously). Depth-averaged velocities for the three conditions are shown in Figure 9.8. Worst-case conditions also were investigated, with a unit river discharge of 130 cfs/ft along the lock wall and both locks discharging for five hours (prototype). Observation of flow patterns indicated no adverse flow conditions in the vicinity of the outlet structure. Model studies to determine stability of riprap to be placed in the vicinity of the outlet structure, Figure 9.9a, indicated that material with a D_{50} size of 24 inches and the gradation shown in Figure 9.9b would be stable for these extreme flow conditions.

The Olmsted project is discussed further in sections 7-2 and 8-22..

9.5 Canal surge and tow squat

<u>Temporary Lock 52, Ohio River</u>. An investigation was made in 1985 of navigation conditions at the temporary 110- by 1200-ft lock, Figure 9.10, constructed at Locks and Dam 52, Ohio River, in 1969 (Maynord, 1987). The new lock is landward of an older 600-ft lock, and normal lock lift is 12 ft. This temporary lock operated for many years without a draft restriction and without damage to the lower miter gate sill. However, there is only 11 ft of depth over the

lower sill, and one pilot, either pushing a heavily loaded tow too fast or with excessive acceleration while over the sill, damaged the lower sill and put the lock out of operation. Following the accident, a draft restriction was strictly enforced. When the gage falls below 10 ft (12 ft of depth over the sill), tows with over 9.25 ft of draft are required to use the 600-ft lock. Drafts of all barges were measured, which increased lockage time and was time consuming. Operators felt that a speed restriction combined with a draft restriction might be more effective.

Early in the study, a limited prototype investigation was made to observe tow movement and to measure speed and squat. Maynord (1987) reports the following observations:

- Towboats operating on the lower Ohio River have a wide range of power, up to 8500 horsepower; larger boats had Kort nozzles with a steering rudder behind the wheel and two backing (flanking) rudders in front. Smaller boats had similar rudders, but open wheels (no Kort nozzles). Connections between the towboat and tow were made in different ways, and there was no consistency in arrangement of empty and loaded barges.
- All pilots used very low headway entering and leaving the lock, with power usually set at 100-200 wheel rpms. Pilots of larger boats cut the power off while the boat was over the lock sill. Very little and very infrequent rudder was applied once the tow was lined up with the lock and sheltered by the approach walls.
- Squat was a maximum (up to 0.8 ft) when the towboat was accelerating or decelerating. While under way at constant speed, squat ranged from 0.1 to 0.65 ft. Squat was less than 0.1 ft when coasting.
- Tows entering the lock from downstream maneuvered slowly until the bow was in the confined section and the tow was aligned with the walls. Tows then came ahead with significant speed.
- In the past, the downsteam culvert valve was often closed after the lower pool elevation was reached in the lock, but operators are now leaving the valve open while tows move in and out of the lock.
- Operators generally lock three tows up and three tows down when tows are waiting.
- Operators stated that some towboats have drafts in excess of 9 ft and that tows often have towboats too small for the load being pushed.

Tow squat is the vertical drop of the tow due to motion, measured from the still water level. Maynord (1987) describes four phenomena causing squat as follows:

a. Displacement squat occurs in confined waterways when water adjacent to the tow is set in motion by displacement of the tow. To maintain the same total energy, the water surface drops an amount equivalent to the kinetic energy of the moving water. It is related to tow speed, ratio of tow cross-sectional area to channel cross-sectional area, and depth of water. Propeller speed is unimportant.

b. Piston squat occurs in locks where the channel is blocked at one end; it is significantly different for tows entering and exiting a lock, Figure 9.11a. Entering tows pile up water in front of the tow, giving them extra depth, and piston squat does not occur. For tows leaving a lock, the volume behind the tow can increase at a greater rate than the return flow under and around the tow, and water depth behind the tow can decrease causing squat. This is not related to propeller movement.

c. Propeller squat is caused by the ability of the towboat to pump water from beneath itself faster than it can be replaced. It is significant only in shallow water and is increased by barges upstream which can block the supply of water to the propellers in a confined waterway such as a lock.

d. Moment squat is caused by the offset between the force produced by the propellers and the force at the connection with the barges, Figure 9.11b. It is greatest with empty barges and produces a moment that tends to force the rear of the towboat down.

Maynord (1987) reported that model studies using both self-propelled tows and a towing apparatus showed that:

a. Squat for entering tows is caused by different parameters than those causing squat for exiting tows. Maximum squat for almost every self-propelled test (entering and exiting) was at the stern of the towboat.

b. For entering tows, tow speed is not important, and displacement, piston, and moment squat were either small or inapplicable. Propeller squat is the primary mechanism producing squat.

c. For exiting loaded tows, propeller squat is an important mechanism. In acceleration tests, during which all tows approached the sill at the same speed, there was increased squat for increased propeller speed.

d. Entry speed can be very irregular due to translation waves from tows moving from unrestricted water into confined water.

e. Unloaded exiting tows can have enough squat to strike the lower sill when operating at high propeller and tow speed and low clearance between tow and sill.

f. Emptying valves should remain open during tow entry and exit. Squat is considerably less with the valves open for equal tow speeds, Figure 9.12a and 9.12b.

g. Large towboats are most likely to strike the lower sill because they have the greatest draft and the greatest potential for producing propeller squat. Small towboats may be susceptible to striking the lower sill because they may have to use increased power while in the vicinity of the sill.

Maynord (1987) pointed out that the primary weakness of the model study was that only one towboat and pilot were used and that the squat/propeller speed/draft relationships for the model towboat cannot be strictly applied to all prototypes. However, identifying propeller speed as the primary variable controlling tow squat in locks can be useful in solving prototype problems.

Bay Springs Lock and Dam. Bay Springs Lock and Dam is the uppermost navigation structure on the Tennessee-Tombigbee Waterway, connecting the two rivers. It is located at the southern end of the Divide Section of the waterway and creates a pool extending through the divide cut to Pickwick Lake on the Tennessee River. The Bay Springs project includes a rock-fill dam, a 110- by 600-ft lock, and a canal extending downstream, Figure 9.13. Bay Springs Lock has a normal lift of 84 ft; maximum lift of 92 ft; and minimum lift of 78-ft. The canal has a 300-ft base width, depth of 13 ft, and is excavated in rock for approximately one mile downstream from the lock, with side slopes of 4V on 1H.

Surge conditions in the canal were investigated in a 1:80 undistorted model (Tate, 1978). A tow consisting of nine barges loaded to 9-ft draft (prototype) was used with a motorized towboat. Design of the original outlet diffuser system is shown in Figure 9.14a. A 1-minute valve opening and 11.9-minute emptying time were used in initial model tests, and this operation produced a 1.9-ft high translatory wave with a steepening leading face which transformed into an undular wave with crests increasing to 2.6 ft above normal pool. Forces measured on tows moored downstream indicated conditions would be very hazardous for navigation. Observations indicated that a tow moving upstream at approach speeds of 2.7 to 4 miles per hr would be transported 60 to 120 ft downstream by the lock release even with increased power applied.

Longer valve opening times were tested, and slower valve opening times significantly reduced wave height and maximum forces exerted on moored tows. A 2-min valve opening time decreased forces approximately 33 percent; valve opening times of 4 and 8 minutes were only slightly better. It was concluded (Tate, 1978) that the undular wave did not form in the model with valve opening times longer than one minute; that the slope of the water surface in the canal rather than wave height was a good indicator of forces on a tow; and that slope of the water surface was a function of speed of valve opening.

The diffuser design was modified, and the design shown in Figure 9.14b was tested. The lock and canal were realigned to place the lock guide wall on the right bank of the canal, permitting tows to use the full width of the canal when maneuvering to enter the lock. The modified design provided a uniform discharge across the width of the canal. Maximum force on a moored tow was reduced from 170 tons to about 40 tons with a 1-min valve opening time and to about 20 tons with a 2-min valve opening time

Studies of the relationship between filling and emptying times for longitudinal floor culvert systems in lock models and prototype indicate that prototype locks will empty about 18 percent faster than the model. The stage-time relation for Bay Springs was adjusted and tested in the model. The expected prototype surge with valve-opening times of 1 and 2 minutes is shown in Figure 9.15. With the 2-min valve opening time, the maximum rate of rise of the water surface was 0.06 ft per sec with a maximum surge height of 2.5 ft above normal pool. Forces on tows did not exceed 36 tons and maintained a uniform rate of loading of approximately one ton per sec.

Based on model tests (Ables 1978), the recommended emptying times for Bay Springs Lock are:

	Filling	Emptying
Valve operating time	1 min	2 min
Model operation	10.5 min	13.3 min
Prototype (estimated)	8.6 min	10.9 min

Details of the emptying manifolds (Ables, 1978) are shown in Figure 9.16.

The intake design with invert at elevation 352, Figure 9.17, was satisfactory and vortex-free. Tests were made also on a 1:25 scale model with the invert of the intake ports raised 8 ft, but the higher level resulted in the formation of persistent swirls over the intake ports. Based on experience, persistent swirls in a 1:25 scale model indicates vortices will occur in the prototype (Ables, 1978).

When too much air is admitted to the filling culverts at the control valves, air pockets form that cause surges when they are released into the lock chamber, and it is, therefore, important to control the admission of air to ensure that only as much air is admitted as can be entrained as small bubbles. The filling valves at Bay Springs Lock were lowered to elevation 304 to obtain desired pressure conditions on the roof of the culvert immediately downstream of the valves during filling operation when cavitation could occur, and controlled air-vent slots in the culvert roof 7 ft downstream of the valve admit air to minimize cavitation. The qualitative effect of air venting is shown on Figure 9.18. Final adjustment of the air vents must be made in the prototype.

Lock and Dam 17, Verdigris River (Choteau Lock and Dam, Arkansas River Navigation Project). Dam 17 is located in the Verdigris River, and Lock 17 is located in a canal about 3400 ft east of the river, Figure 9.19. Normal lift is 21 ft; maximum lift is 24 ft. The upstream canal approach to the lock is 150 ft wide and 9 ft deep for about a mile, Figure 9.19 (Huval, 1980). There are wider reaches, with 300-ft bottom width at the junction of the canal and the Verdigris River and just upstream of the lock to aid navigation and reduce surge effects.

Most towboats operating on the Verdigris at the time this study was made were of the 2000 to 4200 horsepower class, and most tows were about 105 ft wide, with 7 to 8.5 ft draft and about 600 ft long. Such tows occupy a major part of the canal cross section, Figure 9.20, and this causes tows to squat as much as 1.5 to 2 ft below static floating position, depending on tow size and speed. Groundings occurred for both upbound and downbound tows, particularly in the transition reaches. The squat problem worsened when the lock chamber was filled when a downbound tow was in the approach channel. Field tests indicated as much as 1.3 ft of drawdown one mile upstream from the lock (Huval, 1980).

A mathematical model was used to determine lock filling surge heights along the canal for 15 different canal configurations. Results for the maximum surge amplitude near the end of the transition immediately above the lock are summarized in Figure 9.21. For the maximum surge amplitude near the end of the transition immediately above the lock, surge amplitude decreases with increasing canal cross section, and the rate of decrease is greater due to canal widening than to canal deepening. However, clearance under the tow (deepening) is critical at the time of maximum surge.

Tow squat increases as the square of tow speed, and laboratory and field tests indicate that self-propelled tows cannot exceed V_l (Schijf limiting speed) and usually operate at from 50 to 90 percent V_l, Figure 9.22a. The data indicate increasing canal width (and maintaining constant base depth) will not lessen grounding problems for tows proceeding at the highest possible speed

($0.9\ V_l$). It was concluded that widening the canal without deepening probably would aggravate the grounding problem.

The effect of increasing canal depth on tow squat (and maintaining constant base width) is shown in Figure 9.22b. The data indicate that squat at limiting tow speed increases more rapidly with increased depth than with widening. Data in Figure 9.23 indicate that relative squat increases more rapidly by deepening the canal than by widening; however, the increase in squat is small and less than the increase in canal depth. Thus, it is more advantageous to deepen the canal than to widen it for a given increase in cross-sectional area.

It was concluded that a 12- by 300-ft canal cross section would eliminate the possibility of grounding, would significantly improve limiting tow speeds, probably reduce transit times through the canal, and improve navigation conditions.

9.6 Shoaling

In selecting sites for navigation locks and dams on alluvial streams, consideration must be given to sediment transport and deposition patterns. Shoaling in the lower lock approach, if not remedied, can be a serious and continuing problem, expensive for tow operators in lost time and requiring periodic dredging. At sites in bends there is a natural tendency for sediment to be moved away from the concave bank, but special training structures may be required at sites in relatively straight reaches.

The tendency for shoaling (deposition of sediment) in the upstream lock approach can be reduced by constructing ports in the upstream guide wall, with the top of ports below the bottom of the tow and bottom velocities through the ports sufficiently high. Shoaling in the lower approach is a more difficult problem. Sediment moves downstream along the lower lock wall (on the spillway side) and is carried into the lower lock approach as the flow expands downstream at the end of the guard wall and by spillway and power plant flows. Some deposition also occurs due to eddy action in the approach.

Model studies indicated that a properly designed wing dike, extending downstream from the riverward wall for 400 to 600 ft and angled riverward at about 10 degrees, would reduce deposition in the Dardanelle lower lock approach (Figure 9.24 and 9.25). The wing dike, with a top elevation about 2 ft above normal lower pool, permits relatively sediment-free surface flow to pass over the dike while blocking passage of the more heavily sediment-laden bottom currents. Such structures have been effective in reducing dredging requirements at Arkansas River locks, (Franco, 1976).

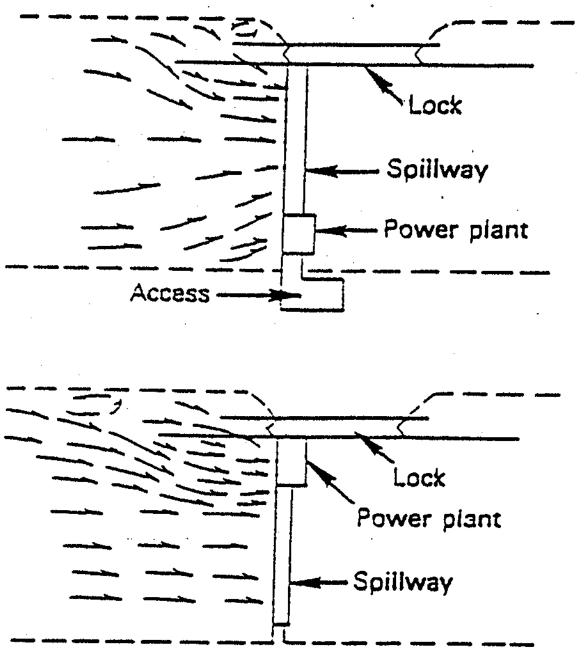


Figure 9.1. Flow patterns in upstream lock approach, Dardanelle Lock and Dam, Arkansas River. (Spillway discharge: 200,000 cfs; power plant discharge: 36,000 cfs.) (Corps of Engineers, 1960).

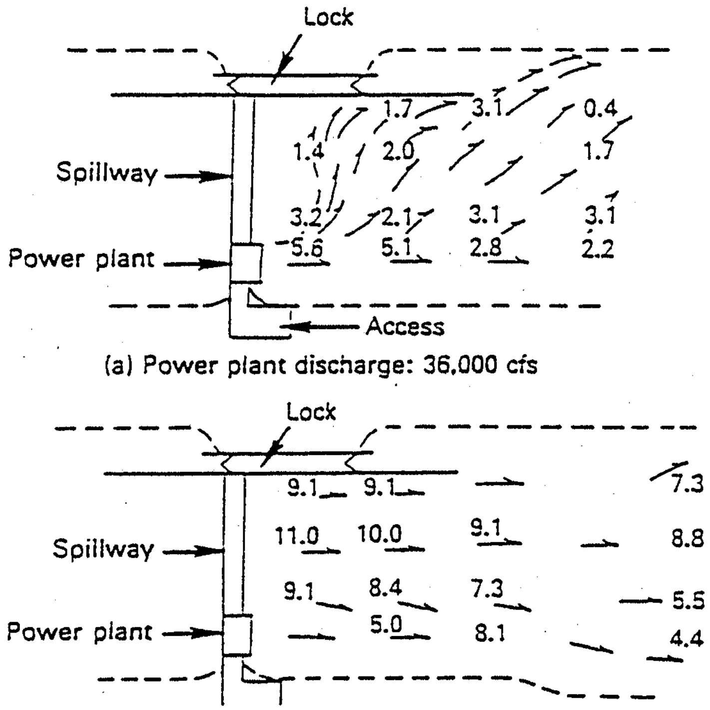
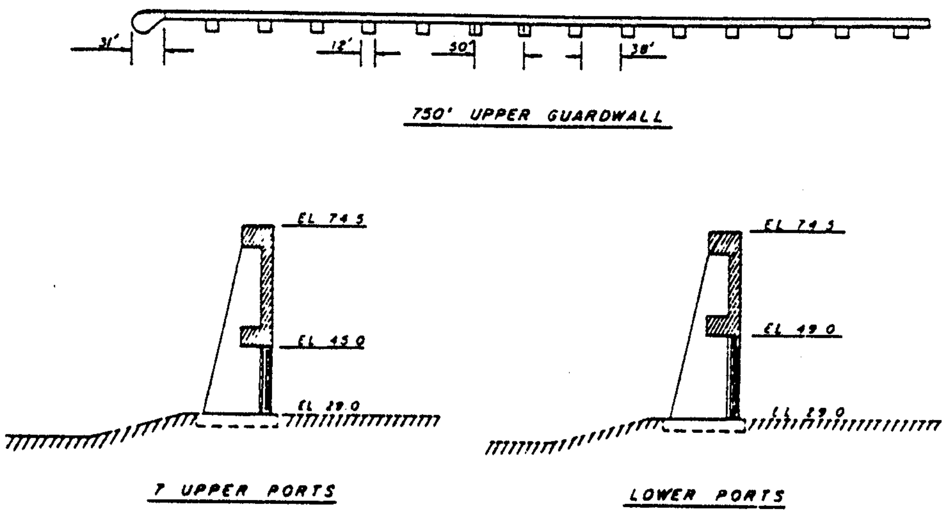
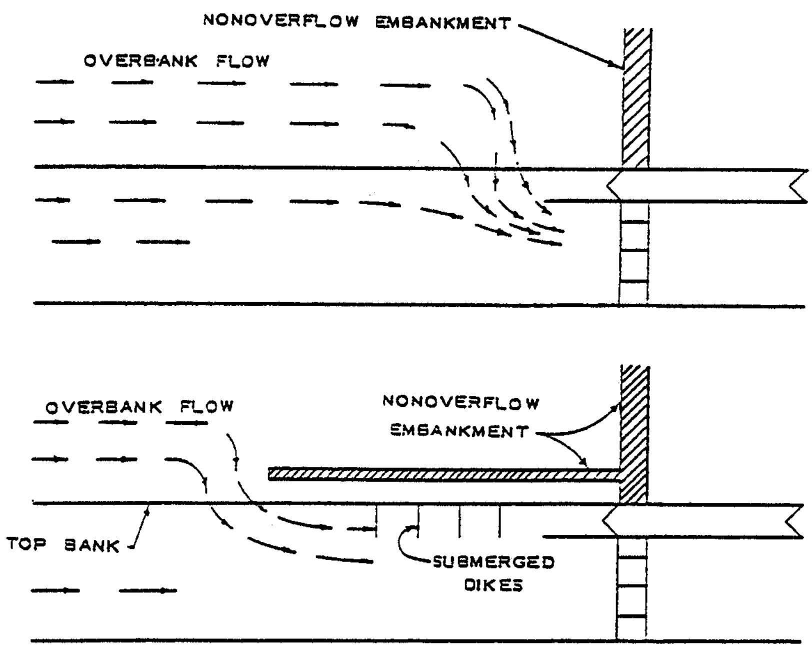


Figure 9.2. Flow patterns and velocities downstream of Dardanelle Lock and Dam, Arkansas River. (Corps of Engineers, 1960)

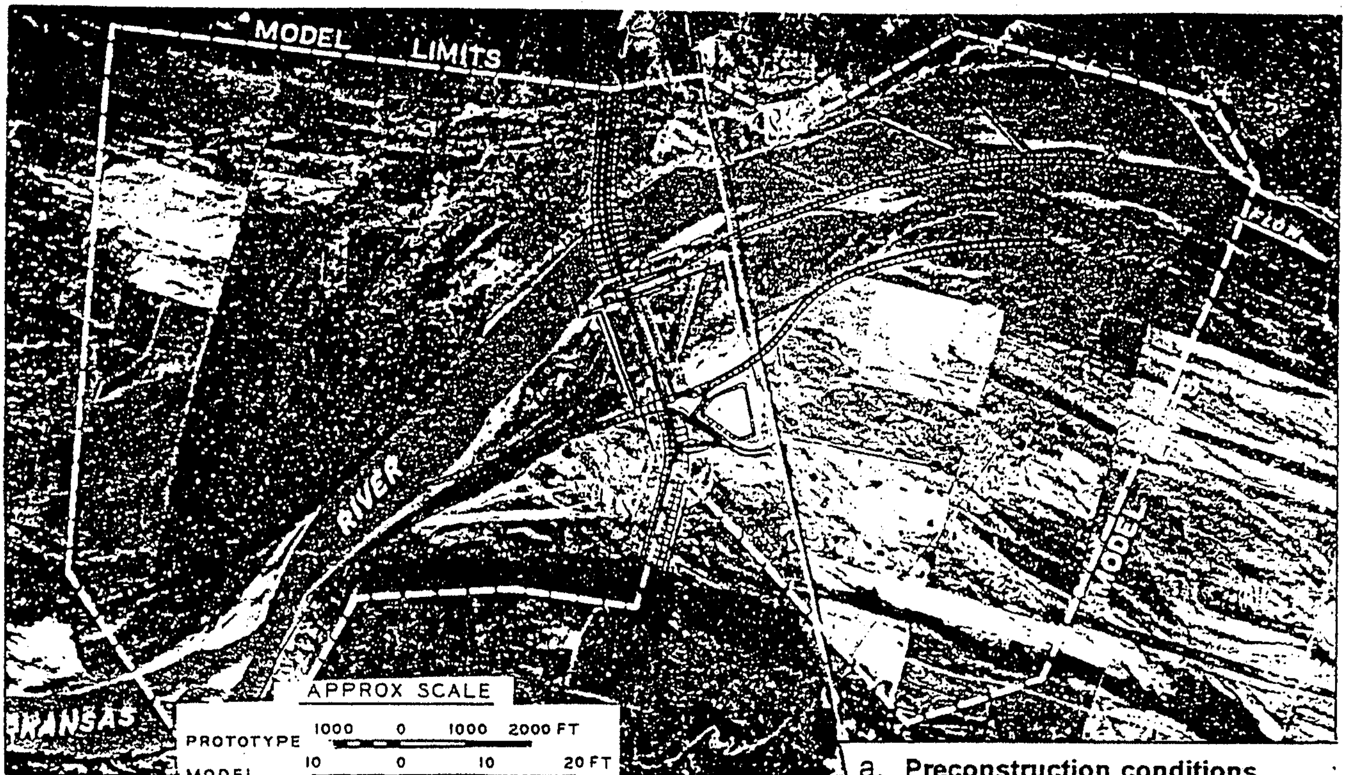


a. Upper guard wall (Shows and Franco, 1979).

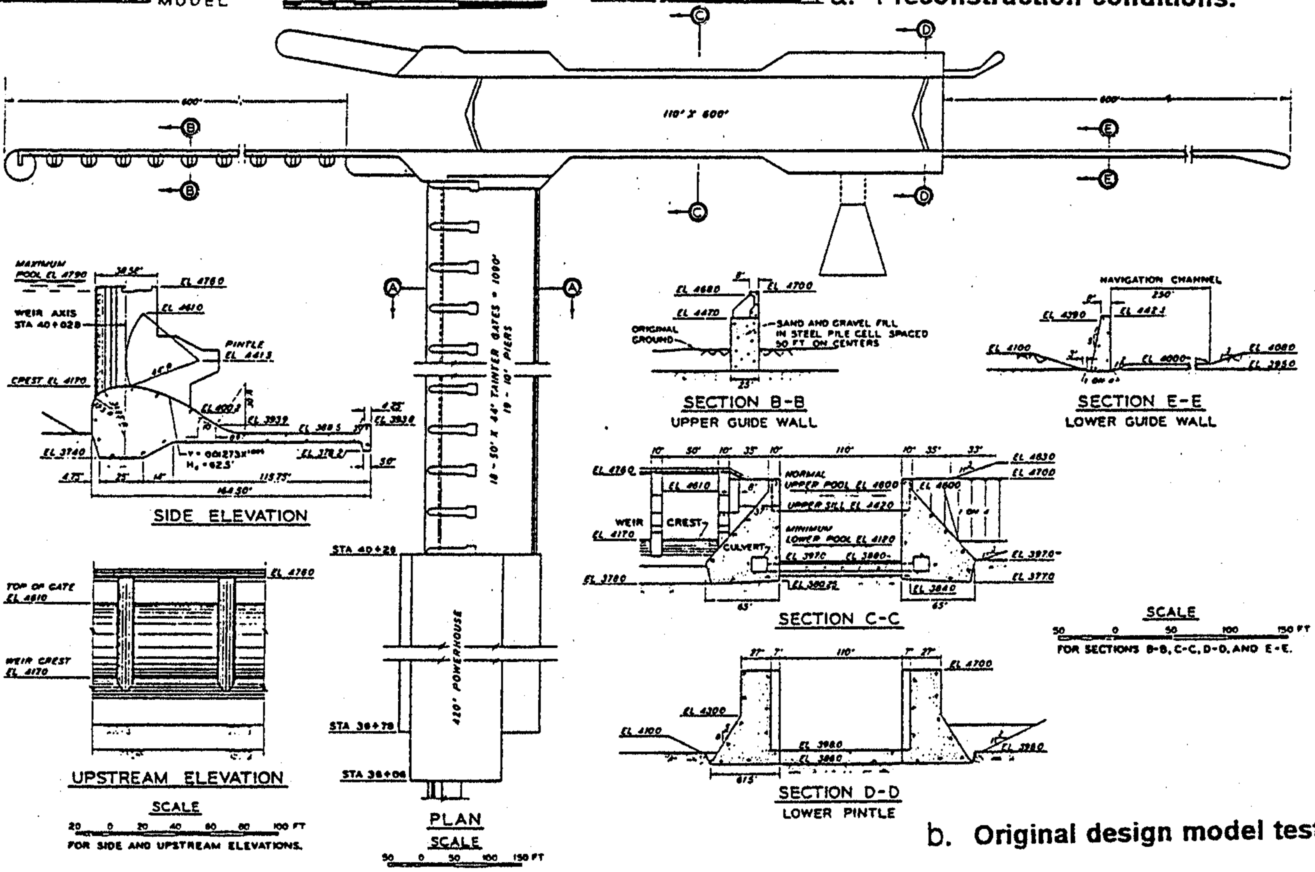


b. Effects of overbank flow and submerged dikes (Corps of Engineers, 1980).

Figure 9.3. Upstream lock approach, Lock and Dam 2, Red River.

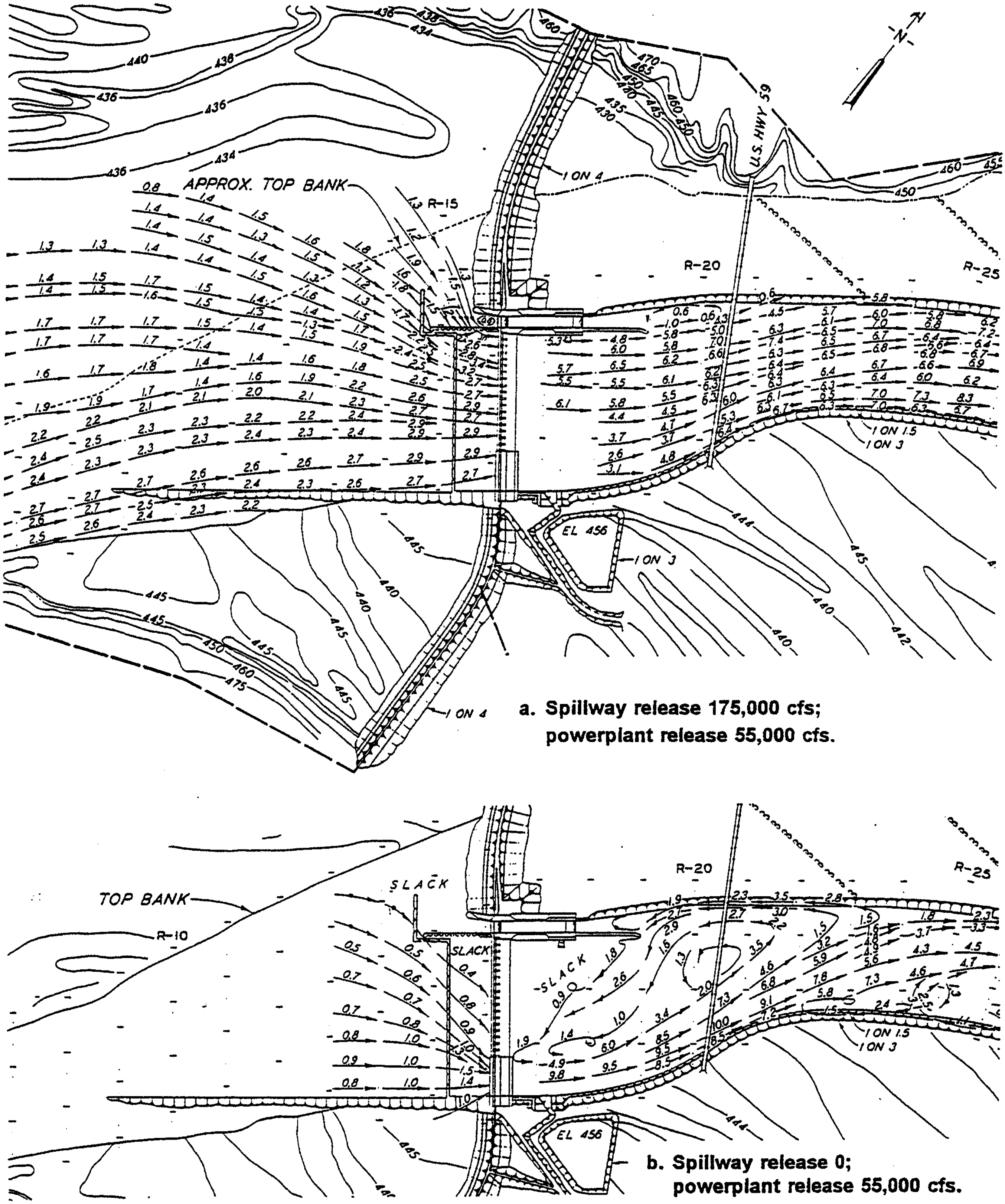


a. Preconstruction conditions.



b. Original design model tested

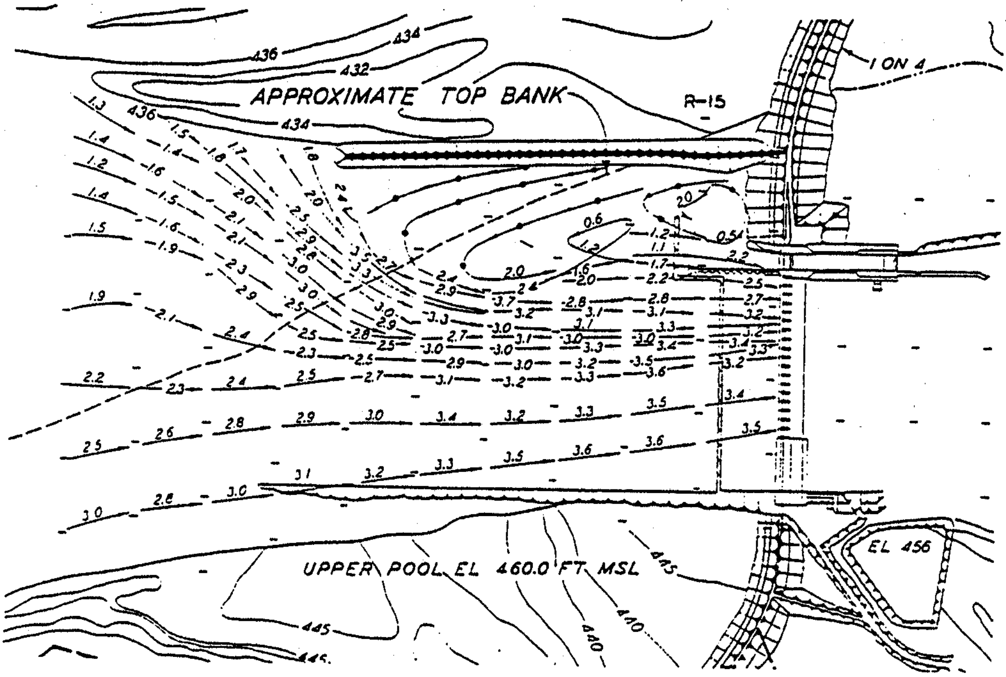
Figure 9.4. Robert S. Kerr Lock and Dam, Arkansas River (Franco and Glover, 1968).



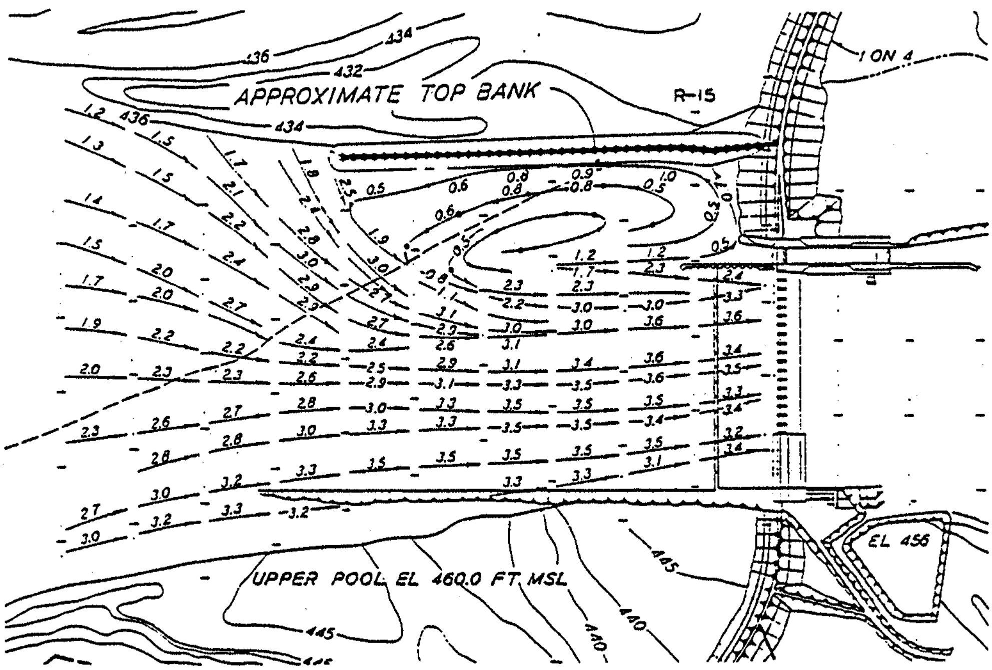
a. Spillway release 175,000 cfs;
powerplant release 55,000 cfs.

b. Spillway release 0;
powerplant release 55,000 cfs.

Figure 9.5. Velocities and currents, original design, Robert S. Kerr Lock and Dam, Arkansas River (Franco and Glover, 1968).

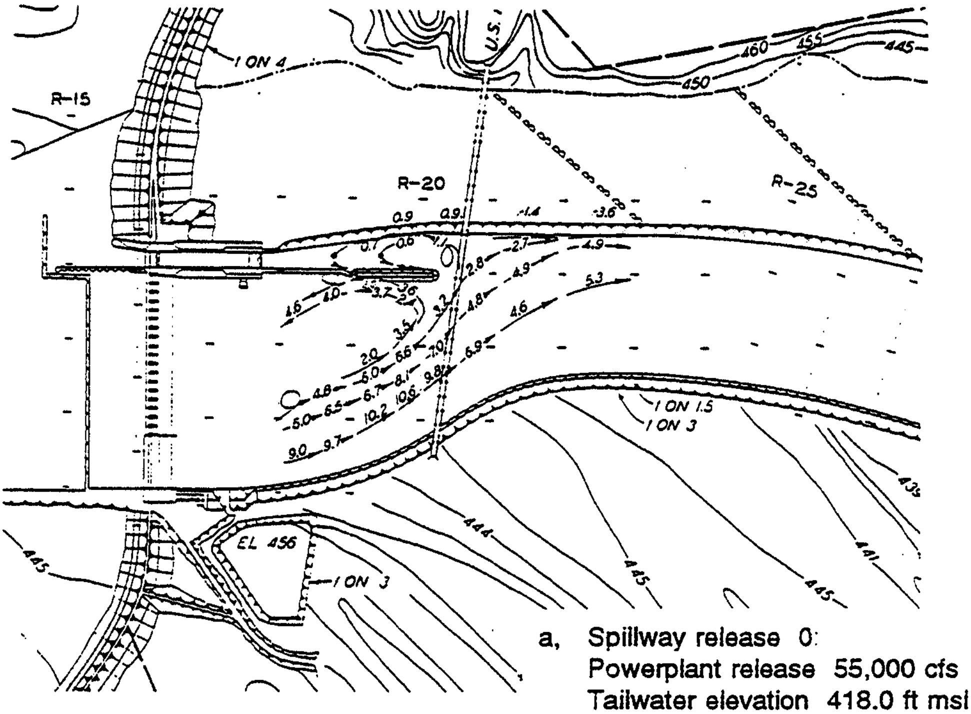


a. Plan C - 3000-ft left bank embankment; ports in upper guide wall.



b. Plan C1 - 3000-ft left embankment; no ports in upper guide wall.

Figure 9.6. Velocities and currents, Plans C and C1, Robert S. Kerr Lock and Dam, Arkansas River. Spillway release 175,000 cfs; powerplant release 55,000 cfs. (Franco and Glover, 1968).



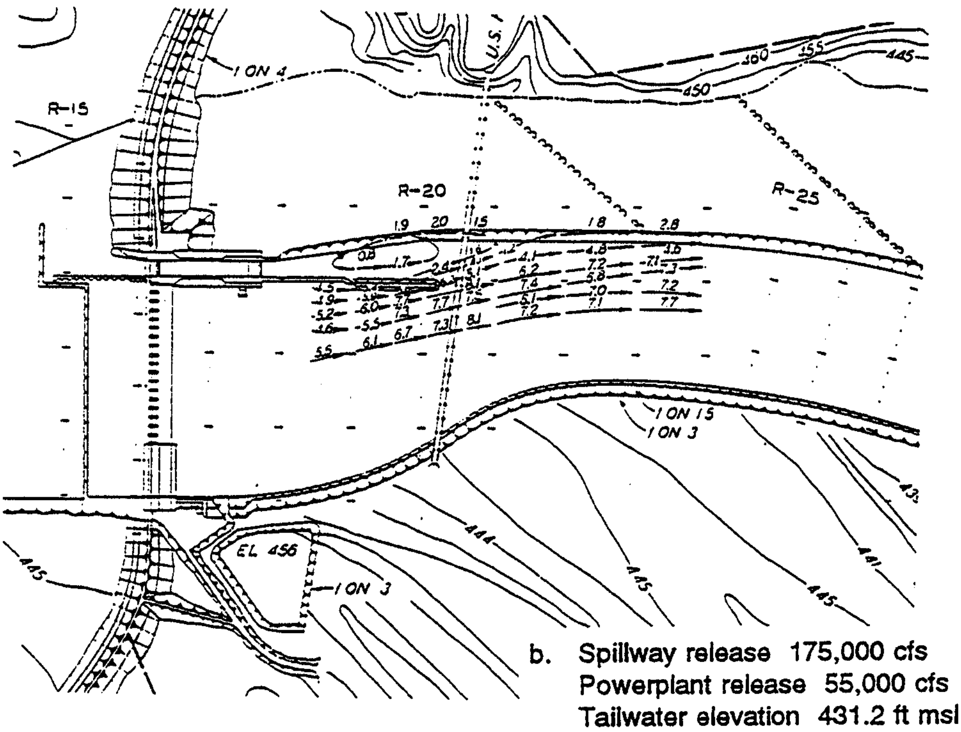


Figure 9.7. Velocities and currents, lower lock approach, with modifications to improve navigation conditions, Robert S. Kerr Lock and Dam, Arkansas River (Franco and Glover, 1968).

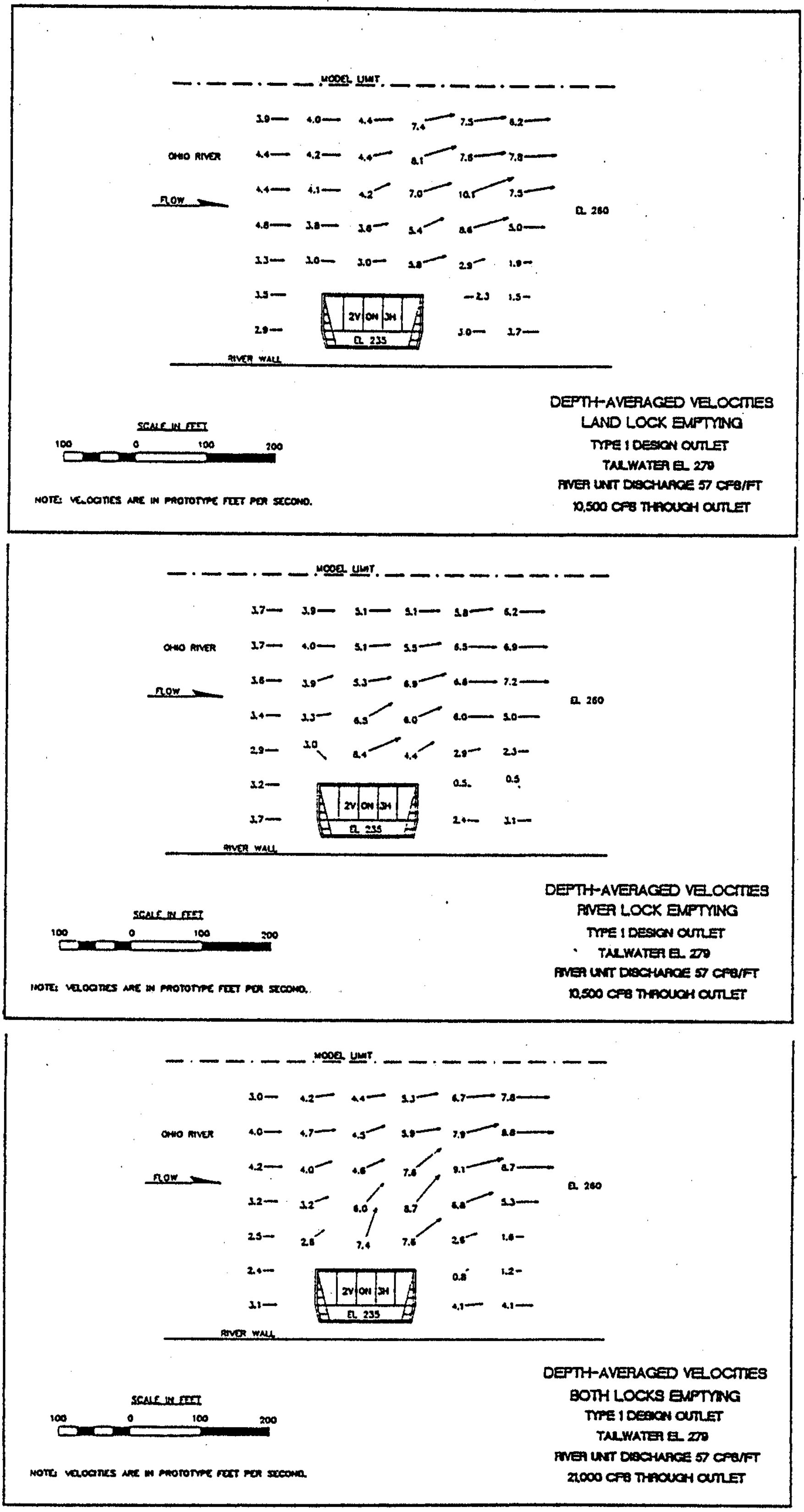
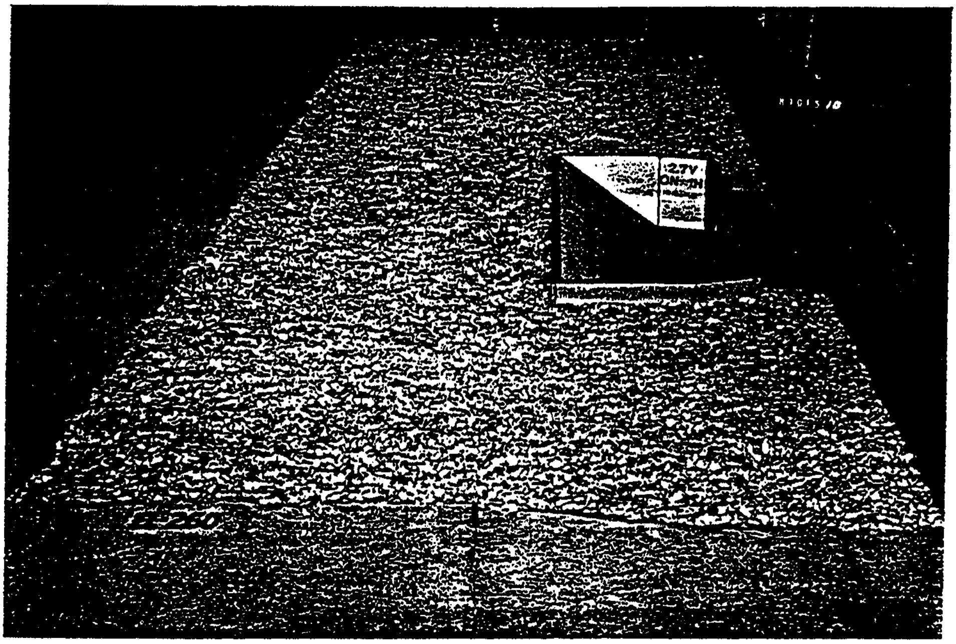
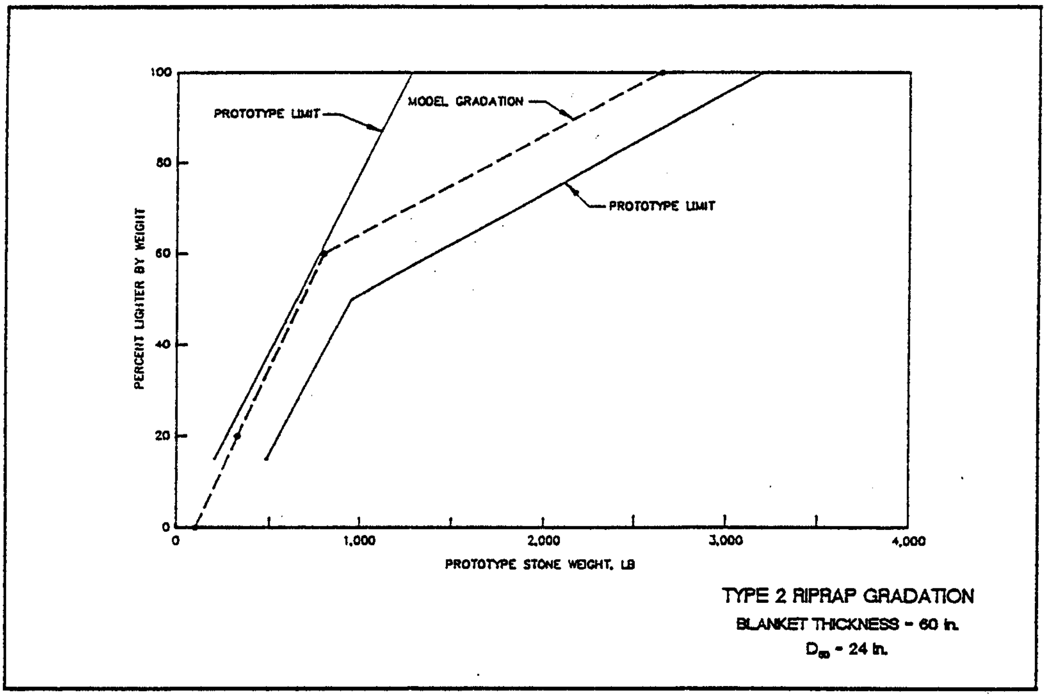


Figure 9.8. Depth-averaged velocities, river outlet, Olmsted Lock and Dam, Ohio River (Stockstill, 1992).

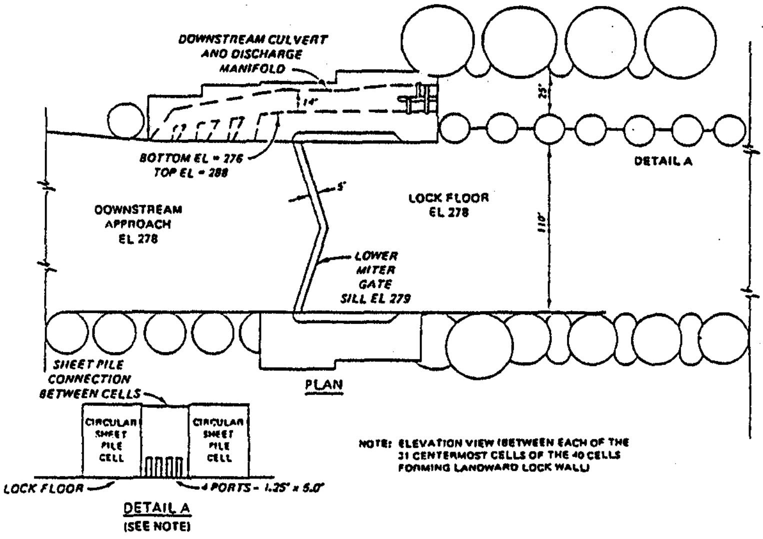


a. Riprap blanket at outlet.

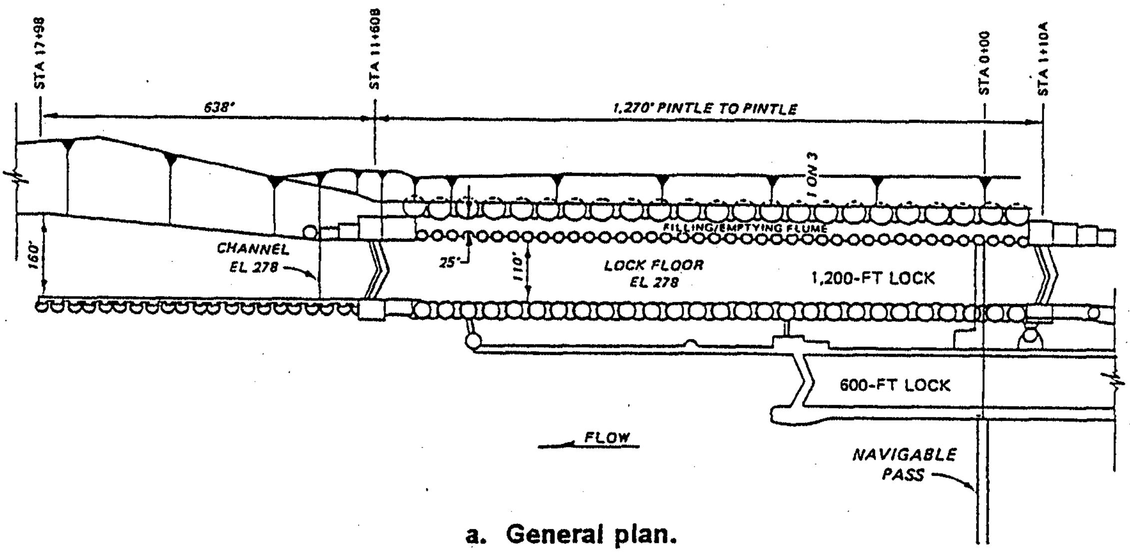


b. Gradation of Type 2 riprap.

Figure 9.9. Riprap protection at river outlet, Olmsted Lock and Dam, Ohio River (Stockstill, 1992).

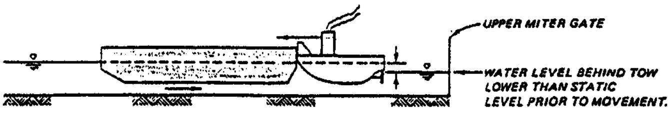


b. Details of lock and discharge culvert.

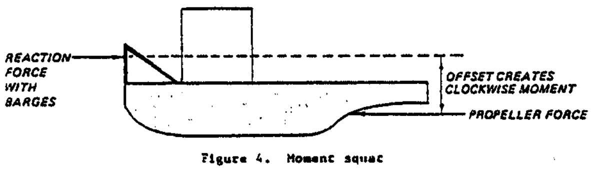


a. General plan.

Figure 9.10. Temporary Lock 52, Ohio River (Maynord, 1987).

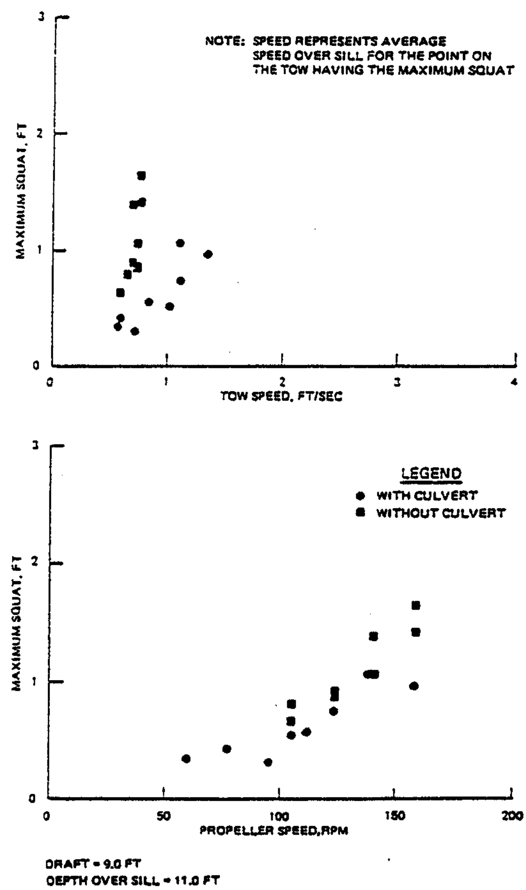


a. Piston squat.



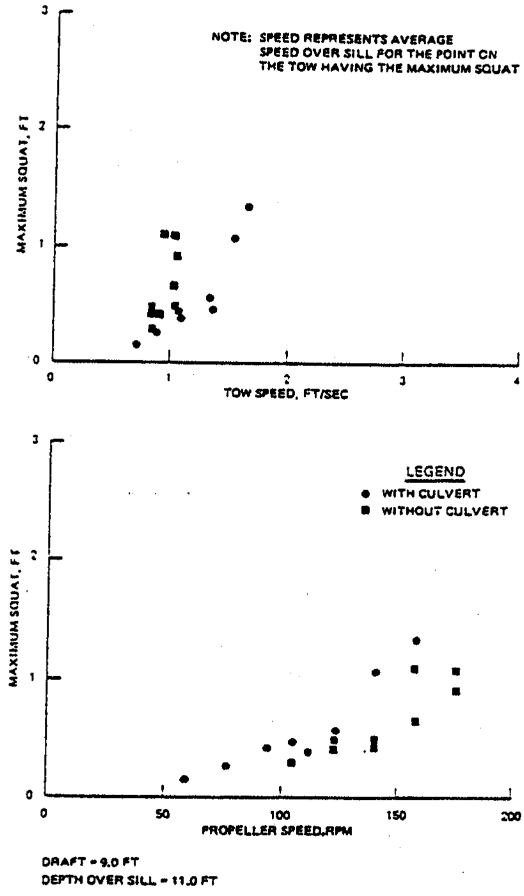
b. Moment squat.

Figure 9.11. Squat mechanisms (Maynord, 1987).



a. Entering tows.

Figure 9.12. Comparison of squat for entering and exiting tows with and without emptying culvert open (Maynord, 1987).



b. Exiting tows.

Figure 9.12. Comparison of squat for entering and exiting tows with and without emptying culvert open (Maynord, 1987).

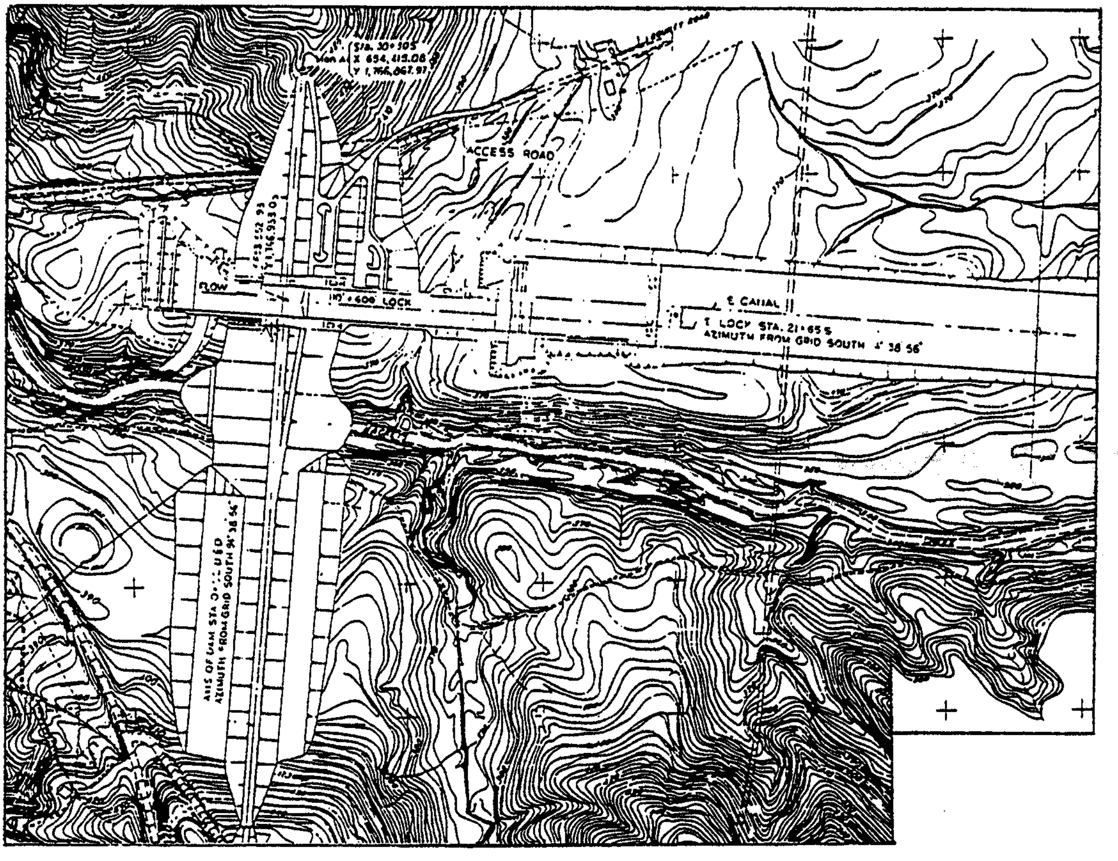
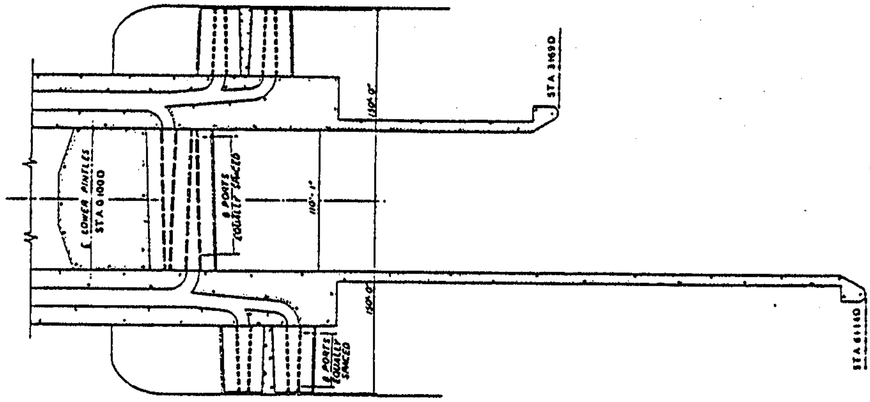
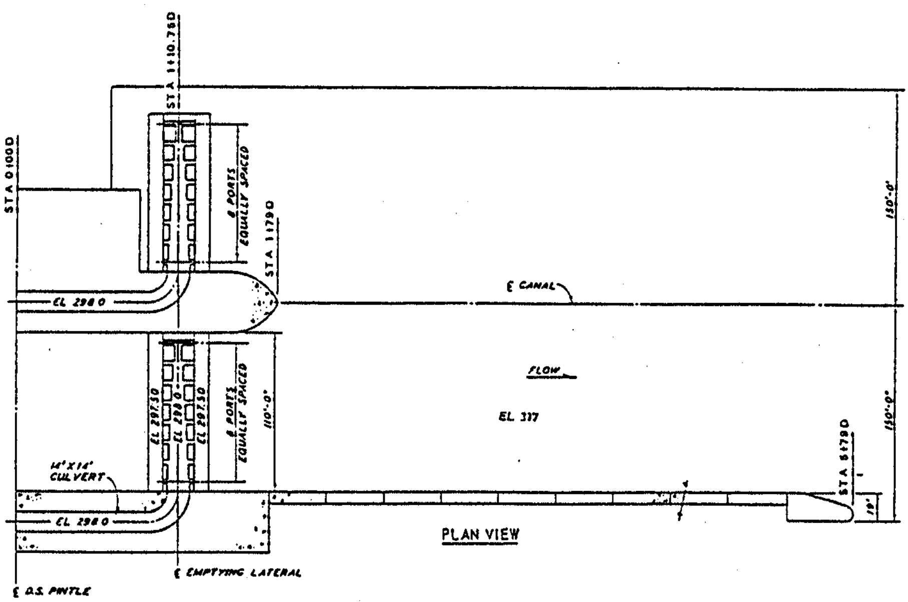


Figure 9.13. Bay Springs Lock and Dam, Tennessee-Tombigbee Waterway, dam, lock and canal alignment (Tate, 1978).

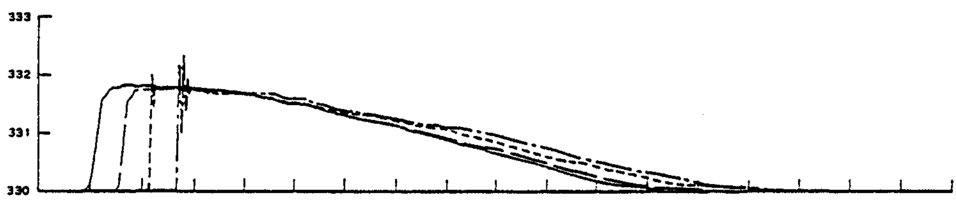


a. Original design.

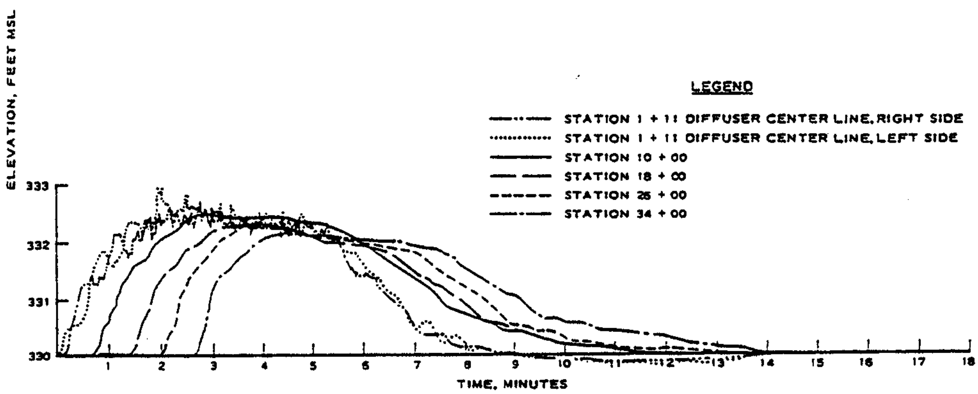


b. Modified design.

Figure 9.14. Outlet diffusers and lower lock approach, Bay Springs Lock (Tate, 1978).



a. 1-minute valve opening time.



b. 2-minute valve opening time.

Figure 9.15. Expected prototype surge, no tow, Bay Springs Lock (Tate, 1978).

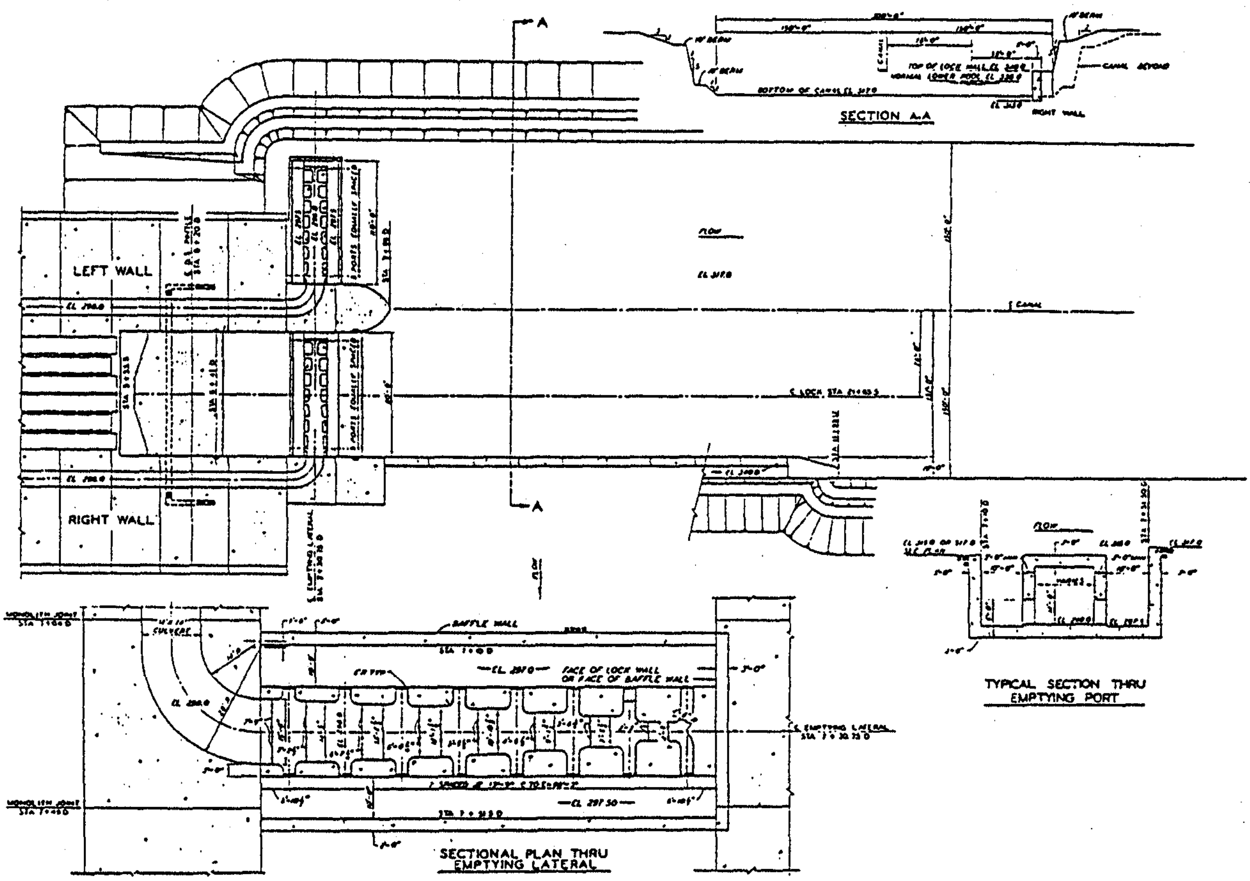
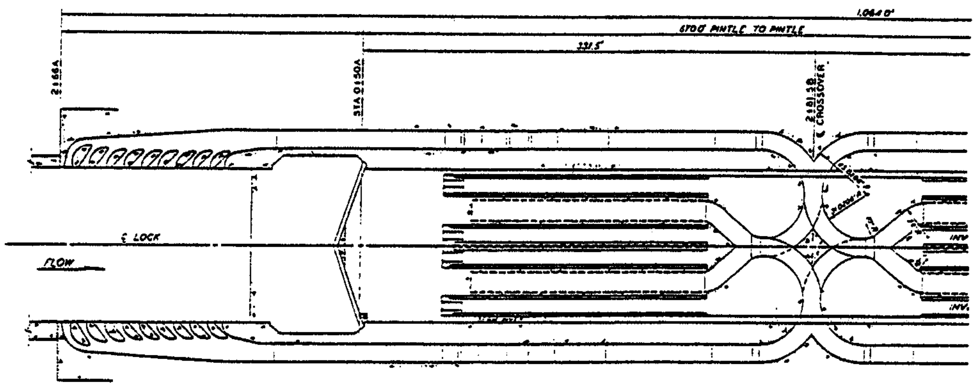
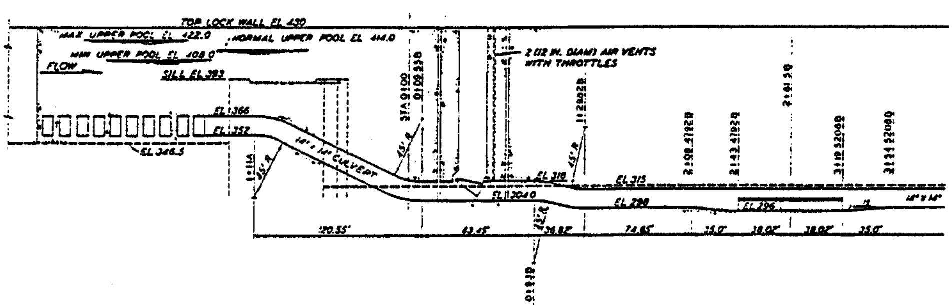


Figure 9.16. Emptying system, Bay Springs Lock (Ables, 1978).



a. Plan.



b. Left culvert elevation.

9.17. Intake system, Bay Springs Lock (Ables, 1978).

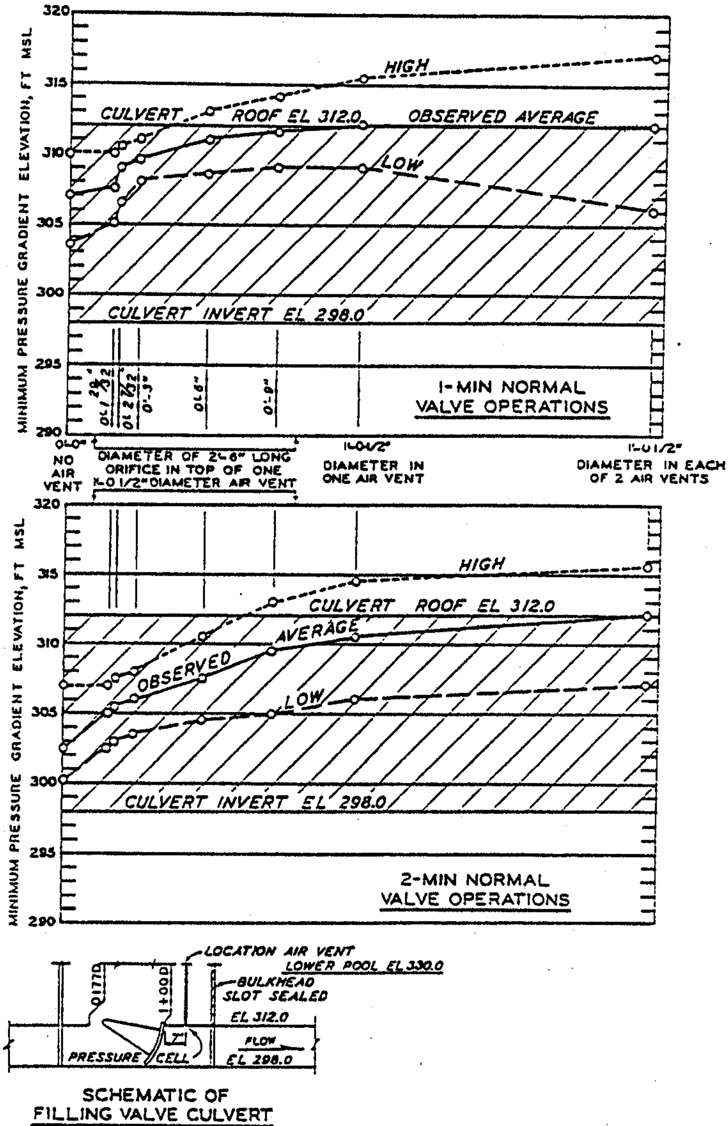
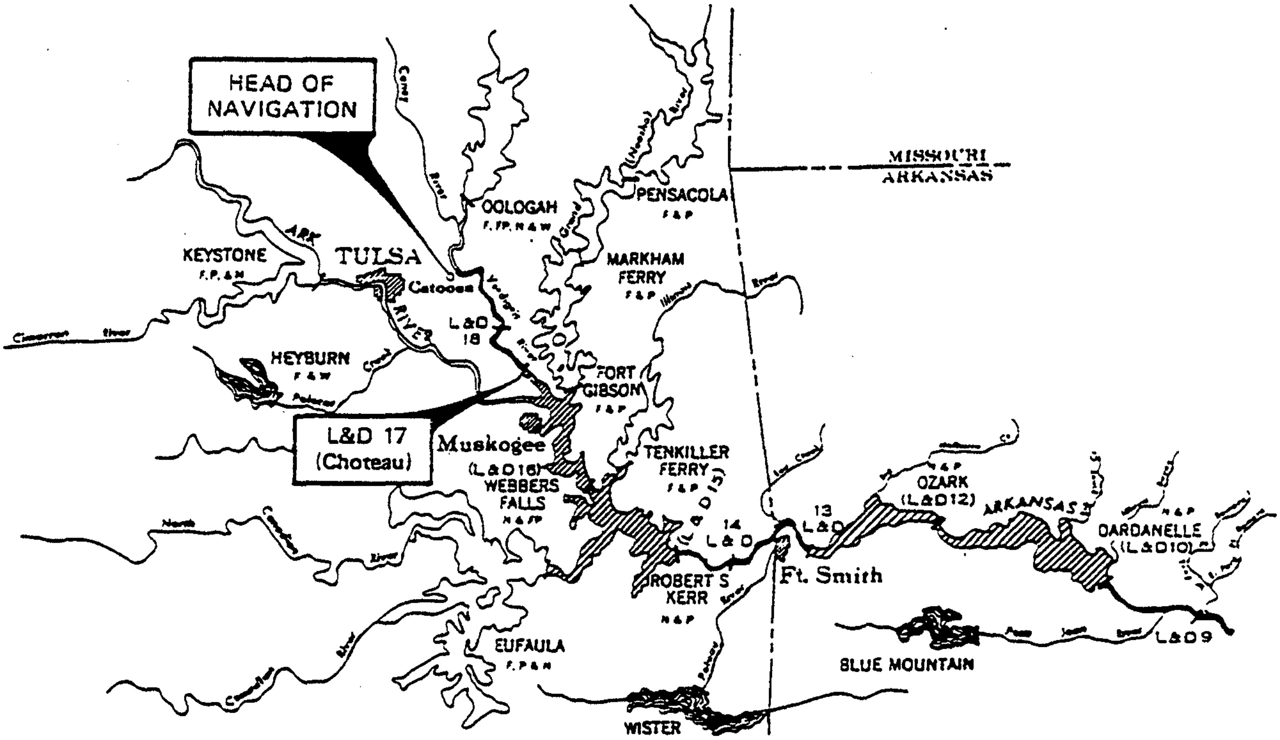
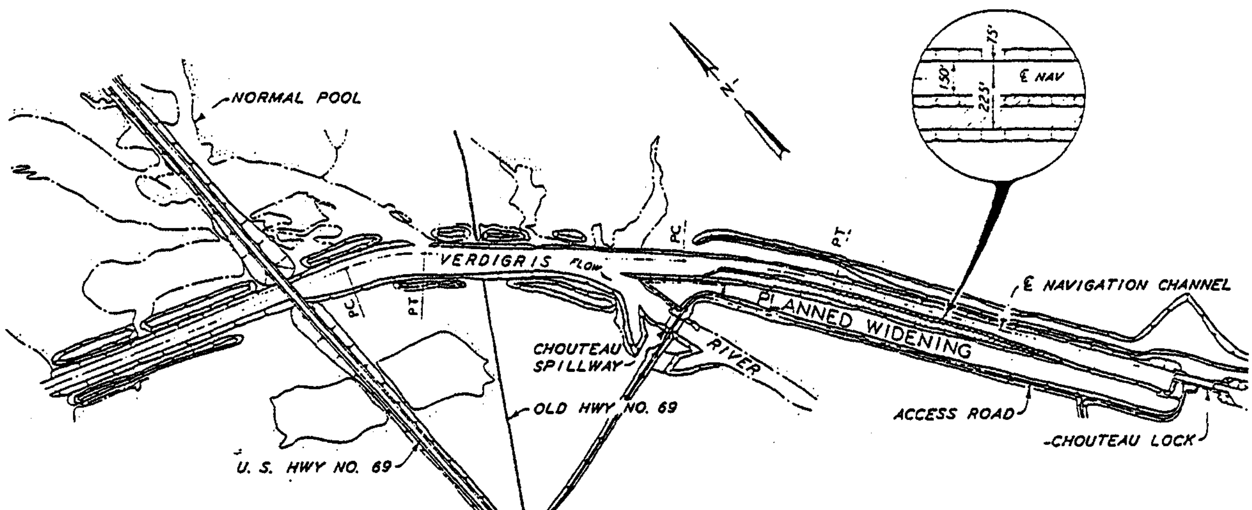


Figure 9.18. Effect of air venting on culvert roof pressures downstream of culvert filling valves, Bay Springs Lock (Ables, 1978).



a. General map.



b. Project layout.

Figure 9.19. Location Maps, Lock and Dam 17, Arkansas River Navigation Project (Huval, 1980).

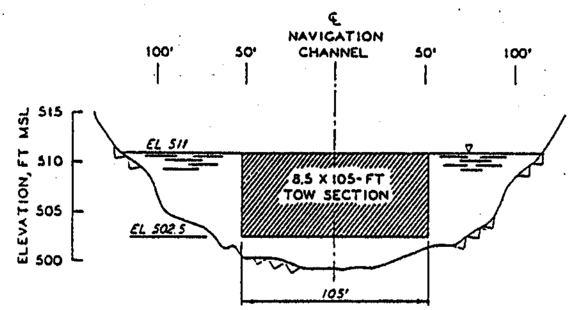


Figure 9.20. Tow in canal, Lock and Dam 17, Arkansas River Navigation Project (Huval, 1980).

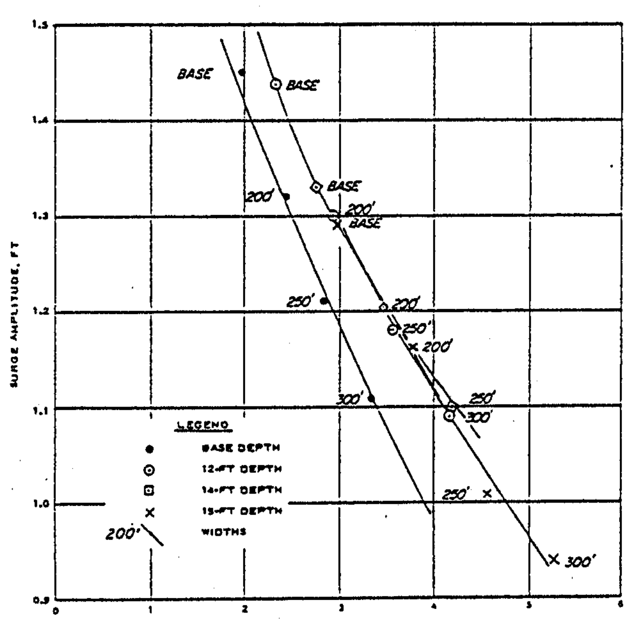
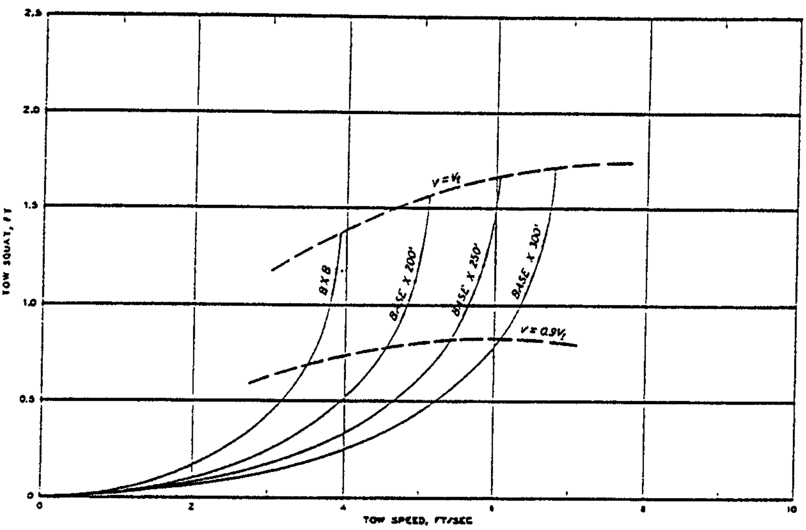
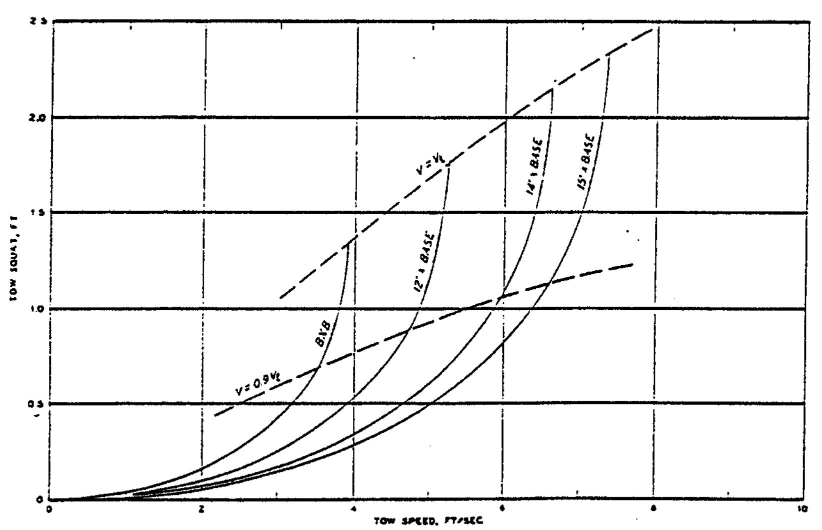


Figure 9.21 Maximum computed surge magnitudes at upstream end of transition above lock (Huval, 1980).



a. Increasing width for constant depth.



b. Increasing depth for constant width.

Figure 9.22. Effect of increasing canal dimensions on tow squat. (Huval, 1980).

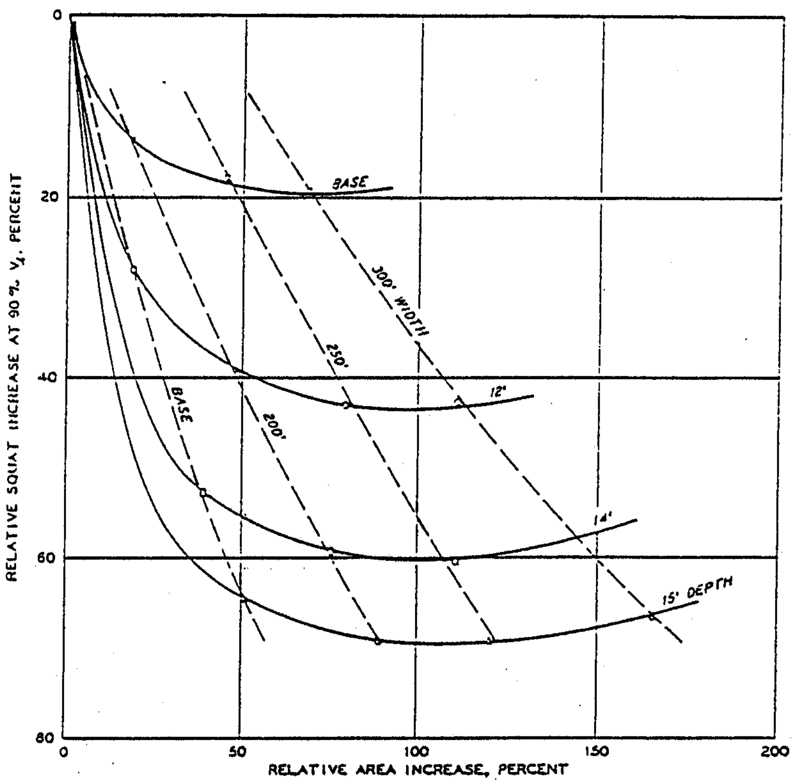


Figure 9.23 Effect of canal size on tow squat at 0.9 V_l (Huvel, 1980).

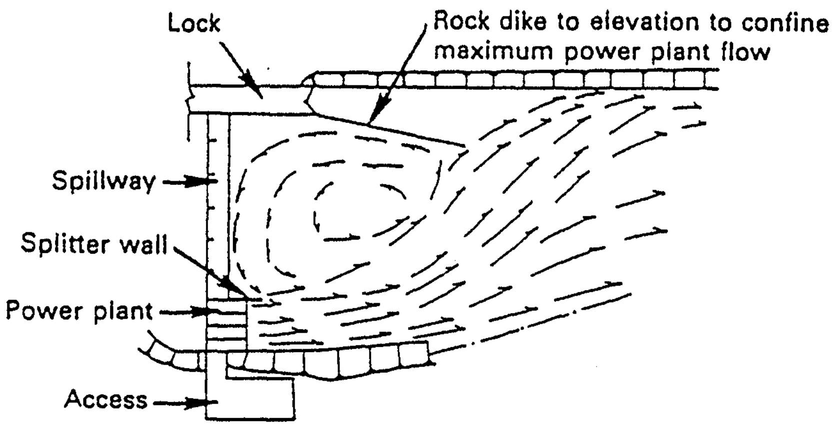


Figure 9.24. Dike to minimize effect of power plant releases on navigation in lower lock approach, Dardanelle Lock and Dam, Arkansas River (Franco, 1976).

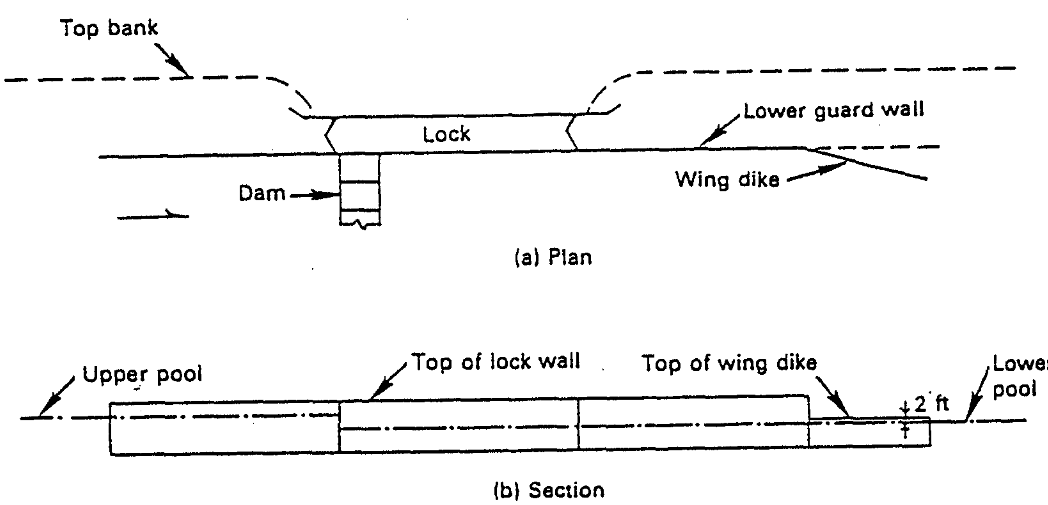


Figure 9.25. Wing dike to minimize shoaling in lower lock approach, Dardanelle Lock, Arkansas River. (Franco, 1976).

10. DREDGING

Dredging is a very costly operation and involves many uncertainties that affect project cost, including a realistic estimate of the total quantity of material to be dredged and characteristics of the material as they relate to the dredge production rate (the rate at which solids are dislodged at the dredging site and transported to the discharge point). Other factors also affect dredging costs. A pipeline dredge operating in a navigation channel may obstruct traffic unless special arrangements are made. Dredges normally operate 24 hours a day, and if the dredge site is in or near an urban area, noise may preclude night operation. Weather conditions may also limit operations under some circumstances.

Costs and potential environmental impacts are fundamental considerations in evaluating alternative dredging and disposal methods and disposal sites. Many factors must be considered in developing a dredging operation, including:

a. Determining the quantity of material to be dredged initially and the frequency and quantity of future maintenance dredging.
b. Sampling to determine the physical and chemical properties of material to be dredged to ensure that the appropriate type of dredge is used, to assess dredge production rates so that time and cost estimates are realistic, and to identify any pollutants in material to be dredged.
c. Selecting the appropriate dredge type and size, disposal method, and disposal area to ensure environmental protection.
d. Identifying adequate disposal areas for both initial and future maintenance dredging, considering the physical and chemical properties of the dredged material.
e. Long-term management of disposal sites to maximize storage volume and beneficial use after the sites are filled.

Dredging for navigation channels is categorized as either initial new construction or maintenance dredging to restore authorized channel dimensions (depths and widths), as follows:

a. Initial construction dredging is dredging to authorized channel dimensions plus an allowance for overdepth dredging to compensate for inaccuracies in the dredging operation.
b. Periodic maintenance dredging is dredging performed on a regular basis, for example annually following the major flood season, to restore authorized dimensions, with the expectation that authorized dimensions will be maintained by the river until the next flood season.
c. Aperiodic or occasional maintenance dredging is done on an "as needed" basis when channel dimensions have diminished to where they limit navigation.

The objective of maintenance dredging is to provide authorized project depth and width at all times in the navigation season. In general, most maintenance dredging in inland waterways is required after river stages fall rapidly on the recession of flood hydrographs, when velocities decrease and coarser sediments are deposited. To minimize delays to shippers, dredging equipment (both government-owned and privately-owned under contract) is available to move quickly to shoaled areas. Priorities in maintenance dredging usually provide that when a number of crossings have shoaled to where navigation is affected, the shallowest crossings are dredged first (when this can be done without excessive movement of dredging plant). This increases usable depth throughout the waterway, and navigation benefits immediately.

In some cases maintenance dredging includes dredging beyond authorized dimensions for "advance maintenance" in critical, fast-shoaling areas, Figure 10.1. Such over-depth dredging can result in lesser overall dredging costs and increases reliability of project depth.

Shoaling and maintenance dredging can be reduced by operating criteria to gradually decrease flood control releases from reservoirs on hydrograph recession. For example, eleven of the upper Arkansas River basin reservoirs are operated to minimize shoaling problems downstream of the navigation locks and dams on the Arkansas River while meeting criteria for releases for flood control, hydropower, and recreation. The operating criteria are complex and are related to season of the year, storage in the flood control space of each reservoir, and storm location and magnitude. The reservoirs are operated so that flood releases gradually taper off on the recession of flood hydrographs and specific targeted rates of discharge reduction are attained at the Van Buren gage. As flood releases drop from 105,000 to 75,000 cfs and from 75,000 to 40,000 cfs, the decrease in flow is limited to not more than 20,000 cfs in 24 hours. From 40,000 to 25,000 cfs, the target taper is a uniform decrease in flow over a 21-day period.

Maintenance dredging can also be reduced by hinged pool operation, as discussed later in this section and in Appendix B.

10.1 Arkansas River Dredging

Alluvial rivers typically follow a meandering, shifting alignment and are wide and shallow. Canalization of such rivers generally requires channel rectification and stabilization work, as well as the construction of navigation locks and dams, to develop a stable channel of adequate navigable depth. Channel training structures are used to modify the curvature of sharp bends and reduce the tendency of the river to shoal in crossings, Figure 10.2. Cutoffs are constructed to eliminate bends of small radius that would be difficult or hazardous for commercial traffic. Such cutoffs usually involve excavating a pilot channel (sometimes by dredging) of small cross section that is widened by action of the river, Figure 10.3. The old bendway is cut off at the upstream end by a closure structure to prevent sediment deposition in the old channel, but remains open at the lower end for recreation access and environmental enhancement.

The lower Arkansas River, in Arkansas, carried a heavy sediment load prior to construction of the Arkansas River navigation project which includes large upstream multipurpose reservoirs that trap much of the sediment load previously transported to the lower river. These reservoirs and stabilization and rectification works were essentially complete prior to construction of Dardanelle Lock and Dam, one of three locks of medium lift (54 ft) on the lower river, Figure 10.4. The upstream reservoirs had already significantly decreased the natural sediment load when construction of Dardanelle was begun in 1959.

Under preproject conditions, the suspended sediment load at Dardanelle averaged 100.4 million tons per year; this was estimated to be reduced to 16 million tons per year under project conditions. It was expected that about 60 percent of the sand load entering Dardanelle reservoir would be deposited in the reservoir, but that 90 percent of the silt/clay load would pass through. In the 13-year period 1965-1977 the average suspended sediment inflow to Dardanelle was about 8 million tons per year, and sediment outflow averaged about 3.5 million tons per year in the period 1964-1981. Operating criteria for the low-lift dams provide for spillway gates to be opened as rapidly as possible on rising stages so that essentially open-river conditions prevail at medium to high flows and the river will retain its sediment transport capacity.

Construction of the low-lift navigation dams began in 1963 with Locks and Dams 1 and 2. By 1968 all the navigation structures were under construction, and the project was completed to Little Rock in December 1968, to Fort Smith in December 1969, and to Catoosa-Tulsa in December 1970. The low-lift navigation dams were sited with the objective of minimizing maintenance dredging at the heads of the pools, and special contraction works were designed for reaches immediately downstream of the locks and dams to aid in providing suitable depths and slopes so as to minimize loss of sediment-transport capacity below the structures.

Pools 9 through 2 downstream from Dardanelle Dam have very different characteristics at normal pool level with regard to:

a. Storage, ranging from 110,000 ac ft at Pool 2 to 32,000 ac ft at Pool 8.
b. Pool length, ranging from 33.2 miles at Pool 2 to 15.8 at Pool 3.
c. Surface area, ranging from 10,500 acres at Pool 2 to 3700 at Pool 3.
d. Average pool depth, ranging from 12.4 ft at Pools 3 and 4 to 7.6 ft at Pool 8.
e. Relationship of normal pool level to the 10,000 cfs flow line.
f. Minimum discharge at which all spillway gates are fully open, ranging from 80,000 cfs at Dam 8 to 280,000 cfs at Dam 2.

All these factors affect the efficiency of stabilization and rectification work in providing a stable navigable channel of adequate depth.

There was significant initial dredging as a part of project construction at the heads of Pools 9 through 2, including 17 million cu yds in Pool 9 immediately below Dardanelle Dam, to hasten development of an equilibrium degraded channel that would provide navigable depth with a minimum of maintenance dredging and meet the scheduled dates for initiating navigation.

Almost all maintenance dredging of the lower Arkansas River has been in the heads of the low-lift pools, at the approach to the next lock upstream, in relatively-long straight reaches, reaches of flat curvature, and long crossings. Except for Pool 2, the bulk of the maintenance dredging was in the early years of project operation prior to 1976. Shoal areas at the head end of Pool 2, Figure 10.5, are representative of areas requiring maintenance dredging. Pool 2 has had the highest rate of dredging of all pools and accounted for 36, 63, and 72 percent of all maintenance dredging in Arkansas in 1973, 1986, and 1993, respectively.

Additional contraction was added in some reaches of the pools after the project went into operation to minimize maintenance dredging and provide more reliable navigable depth. The authorized channel depth in the Arkansas River is nine feet; authorized channel width is 250 ft at project depth.

Maintenance dredging was negligible in the 1978-1984 period in Pools 9 through 3, averaging about 150,000 cu yds per year. Studies indicated that deposition in Pool 2, where maintenance dredging averaged 430,000 cu yds per year in the 1978-1984 period, is probably more related to pool characteristics than to design of the stabilization and contraction works. Pool 2 is significantly longer and has more storage at normal pool level than Pools 9 through 3, and it is subject to open-river flow conditions more rarely than the other pools (spillway gates fully open about once in seven years, on the average, compared to annually at the other pools) (Petersen and Laursen, 1986).

Schmidgall (1981, 1985) examined maintenance dredging on the Arkansas River as related to streamflow and concluded that the amount of dredging required in most pools is related to volume of flow. His data relating annual maintenance dredging in the State of Arkansas (the lower reach of Pool 13 through Pool 2) to annual streamflow at Van Buren are shown in Figure 10.6a. His data for the total system for 1969 through 1994, shown in Figure 10.6b, indicate that maintenance dredging has decreased significantly with time.

Cumulative dredging volume from when the project became operational in 1969 through 1984 is shown in Figure 10.7 for two reaches: Pools 9 through 3 and Pools 9 through 2. Data in the figure indicate that, if one disregards dredging in Pool 2 (on the basis that it is atypical of pools downstream of Dardanelle), annual dredging decreased significantly with time over the period of study, and was at a relatively constant and negligible rate of 780 cu yds per 100,000 ac ft of flow (or 150,000 cu yds per year) for the period 1978-1984. The data also clearly suggest that deposition problems in Pool 2 are of a different order of magnitude (and probably of different origin) than those in Pools 9 through 3.

Maintenance dredging on the Arkansas is initiated whenever depths in the navigation channel become less than the authorized 9-ft depth. Typically, maintenance dredging is to a depth of 12 ft, including 3 ft of overdepth dredging for advance maintenance to allow a time period for sediment buildup before the 9-ft authorized depth is no longer available and maintenance dredging must be repeated. The objective is to provide authorized navigable depth 100 percent of the time to the extent feasible. Dredging typically begins on the hydrograph recession at flows in the order of 120,000 to 70,000 cfs (flows that carry a significant sediment load with depths considerably in excess of authorized depth) to minimize potential interruption of navigation.

The Little Rock District awards two maintenance dredging contracts in January each year for work in the calendar year, and contracts run concurrently. Two cutterhead dredges are used, one assigned to Russellville (Dardanelle area) and the other assigned to Pine Bluff, but both work in any area of the river, as needed. In the period 1979 through 1989 (including three years in which the flow volume exceeded 30 million acre-ft), dredging in Arkansas ranged from 329,000 yds^3/yr in 1980 to 5,953,000 in 1988 and averaged 1.94 million yds^3/yr. In the period 1984 through 1994 (including six years in which the flow volume exceeded 30 million acre-ft), dredging in Arkansas ranged from 1,314 million yds^3/yr in 1984 to 4,785 in 1988 and averaged 2.27 million yds^3/yr.

10.2 Mississippi River Dredging

The Mississippi River has a navigable length of 1811 miles. Authorized channel dimensions are 9 by 150 ft from miles 857.6 to 853.4; 9 by 200 ft from miles from 853.4 to 815.2; and 9 by 300 ft downstream through the Vicksburg District. (Mileage above the mouth of the Ohio River at Cairo is measured as "miles above Cairo;" mileage below the confluence with the Ohio is measured as "miles above Head of Passes" at the mouth of the Mississippi River.) The river is canalized downstream to the vicinity of St. Louis, Figure 10.8.

The St. Paul District, Corps of Engineers, is responsible for the Upper Mississippi River downstream to below Lock and Dam 10 (mile 857.6 to mile 614). Dredging is accomplished with a 24-in cutterhead dredge owned by the District (and loaned for work in other Districts as well) and through annual one-year contracts with firms using mechanical draglines. In the 1975-1989 period, maintenance dredging averaged 750,000 cu yds per year, 600,000 by the cutterhead dredge and 150,000 by contract.

The Rock Island District of the Corps is responsible for the Upper Mississippi from just below Lock and Dam 10 to just below Lock and Dam 22 (mile 614 to mile 300). Most maintenance dredging is accomplished using the 24-in cutterhead dredge owned by the St. Paul District. In the 1986-1989 period, maintenance dredging averaged 570,000 cu yds per year. In 1989, 572,000 cu yds of material was removed from nine sites with the cutterhead and 29,400 cu yds were removed mechanically by dragline and clamshell dredges under contract.

The St. Louis District overseas the river from just below Lock and Dam 22 downstream to the mouth of the Ohio River (mile 300 to mile 0). There are four navigation locks and dams in the upper 100 miles of this reach, and open-river navigation prevails downstream. In the past up to 12 dredges were used in the St. Louis District for maintenance dredging, but currently only two are used routinely. One is a dustpan dredge owned by the District that usually works in the open-river reach. A cutterhead dredge is under contract from a private firm to dredge the navigation pools. The contracts are for one year and are paid on a per-cu-yd basis. Typically, $7 million to $8 million is spent on dredging each year (at $0.,75 to $1.00 per cu yd). However, because of the severe drought and record low stages in 1988 and 1989, approximately $23 million was spent in each year, and six additional dredges were required. Four were contracted from private firms, one was borrowed from the Memphis District, and one was borrowed from the St. Paul District (Derrick, 1991).

The Memphis District oversees the river from the confluence of the Ohio and Mississippi Rivers (mile 953.8) downstream to the mouth of the White River (mile 599). In the 1985-1989 period, maintenance dredging averaged 28.2 million cu yds per year. Work is accomplished by four dustpan dredges, three of which are Corps-owned and one under a year-round rental contract. These dredges are used where needed in the St. Louis, Memphis, Vicksburg, and New Orleans Districts.

The Vicksburg District is responsible for the reach of the Lower Mississippi River from the mouth of the White River downstream to just above the Old River Control Structure (mile 599 to mile 320.6). The Vicksburg District uses a combination of revetments, dikes, and dredging to maintain the navigation channel. From 1984 to 1989 an average of 2,285,000 cu yd of material was dredged each year. Most dredging is performed using two dustpan dredges, one owned by the District (and on loan to New Orleans District much of each year) and the other under a year-round contract with a private contractor.

10.3 Missouri River Dredging

Construction of six mainstream dams on the upper Missouri River has reduced the average sediment load from 200 to 50 millions tons per year, with an increase in the percentage of sand load and a decrease in percentage of silt and clay load. The Missouri River is an open-river waterway, and there has been no maintenance dredging in the navigation channel above Rulo since 1969. In the Kansas City District, below Rulo, the channel is contracted by dikes and no dredging was performed between 1980 and 1988. However, severe drought necessitated reservoir releases to be cut back below normal levels in 1988, 1989, and 1990, and depths dropped to less than the authorized 9-ft project depth. Approximately $775,000 worth of dredging was done in 1988 and 1989 using a cutterhead dredge borrowed from the St. Paul District.

10.4 Red River Dredging

Five navigation locks and dams were recently constructed on the lower Red River, as discussed in Appendix B. Lock and Dam 1, Figure 10.9, was completed in the fall of 1984, and significant sediment problems were experienced at the lock shortly after the project went into operation. Channel expansion and flow separation created slack water conditions and eddies at the lock and dam. Studies indicated that structural measures were required to either reduce the amount of sediment deposition or relocate it into more manageable (more easily dredged) areas. These measures included construction of dikes in the upstream lock approach channel and raising the wall that separates the downstream lock approach from the main channel. Periodic deposition has still occurred after these modifications were made, but it is to a much lesser extent than previously and in areas that can be easily dredged.

An unusual aspect of the deposition at Lock 1 is that deposition has occurred in the vicinity of the miter gates, Figure 10.10. In 1990, Vicksburg District rented an 8-in submersible pump for trial use in removing sediment in the vicinity of the miter gates. The material removed was fine sand and silt that, when compacted, becomes very hard and difficult to remove. The pump was used at three locations. The first test site was an area about 85 ft wide by 12 ft in the downstream direction, and about 6 ft deep downstream from the lower miter gates where material had settled out during spring 1990 high water; material was removed to prevent problems in opening and closing the lower miter gates. The second test area was inside the lock just upstream of the lower miter gate, measuring 48 by 85 ft and about 4 ft deep. The material had been compacted by currents, and opening and closing of the miter gates made the material very dense and hard. Pump production rate at these two sites was about 60 cu yds per hr. The third test site was between the downstream guide wall and the "I" wall where the material was clean sand, and the production rate was about 300 cu yds per hour.

As a result of success with the leased pump, the Vicksburg District purchased a 10-in submersible pump in 1991. Neilans, et al. (1993) report that the pump was used about three times a year at each of the three lower locks on the Red River, taking between 2 and 3 days to remove the sediment buildup from each lock.

An advantage of the submersible pump is quick response time. When clearing is needed, the submersible pump can be deployed in about four hours if the District's towboat is available. Maneuverability of the submersible pump makes it particularly well suited for removing sediment around the miter gates because it can be positioned in corners and along walls without damaging either the lock or the equipment.

10.5 Effect of Hinged Pool Operation on Maintenance Dredging

Hinged pool operation is a spillway gate operational procedure designed to lower normal upper pool level at a lock and dam to increase velocities through the deeper downstream reaches of the pool, with the objective of decreasing maintenance dredging requirements by moving depositing sediments farther downstream in the pool and lessening deposition at the head of the pool, as discussed further in Appendix B.

Locks on the Arkansas River were designed for hinged pool operation and have upper miter gate sills set low enough for tows to enter the locks with the upper pool drawn down five feet below the normal navigation pool level. This drawdown at the dam decreases depths and increases velocities through the downstream reach of a pool, thus moving depositing sediments farther downstream into the deeper reaches of the pool. On flood recession, after most sediments have settled out, the normal navigation pool is re-established. Water depths over the sediments deposited in the downstream reach of a pool are adequate to support navigation without dredging.

Hinged pool operation has been tried at most Arkansas River dams with various degrees of success. In the most successful hinged pool operations, the water level was drawn down only 2 or 3 ft, rather than the full design drop of 5 ft. Good results were achieved in moving sediments through the navigation channel in the upper reaches of Dardanelle Lake during recession of the 1995 floods by using a 2-ft drawdown hinge.

10.6 Dredging Equipment

Modern dredge plant can be classified as either mechanical or hydraulic (or a combination of the two). Mechanical dredges lift the dredged material by means of diggers or buckets of various design, and hydraulic (suction) dredges pick up material by means of suction pipes and pumps.

Mechanical Dredges. Mechanical dredges remove loose soft or hard materials by a dipper or bucket of some type and usually operate in conjunction with disposal barges that are filled with the excavated material and then moved to a disposal site and emptied. Dipper and bucket dredges are similar in that both operate with the dipper and bucket at the end of a boom, but the dipper is rigidly attached to the boom and the buckets are suspended by cables, Figure 10.11. Bucket and ladder dredges dig the material out using a chain of buckets rotating around a ladder, with the buckets discharging onto a conveyer belt that moves the dredged material to the disposal barge or site. These dredges are not usually self-propelled, but are moved to the work site by a tow. They can maneuver in a limited area by using spuds (Figure 10.11.)

Hydraulic Suction Dredges. Hydraulic suction dredges are usually categorized according to the means of disposal of the dredged material (hopper, pipeline, and sidecasting dredges) or according to the means for picking up the dredged material (cutterhead, plain suction, and dustpan dredges).

- **Hopper dredges**, Figure 10.12, are deep-draft seagoing vessels used primarily for work in exposed harbors and shipping channels where traffic precludes use of stationary pipeline dredges. They are not used in shallow-draft waterways in the United States.

- **Sidecasting dredges** are self-propelled shallow-draft seagoing vessels designed for dredging from bar channels at small coastal harbors that are too shallow for hopper dredges and too rough for pipeline dredges to operate. A sidecasting dredge picks up bottom material through two suction pipes and discharges it directly overboard outside the channel prism through a discharge pipe.

- **Hydraulic pipeline dredges** draw a slurry of bottom material and water through a suction line and pump the slurry through a floating discharge line to the disposal site. They are of three types: dredges with a plain suction intake, dredges with a cutterhead at the forward end of the suction line to loosen material to be dredged, and dustpan dredges with jets in the head to loosen material.

Cutterhead dredges, Figure 10.13, are the most widely used type in the United States and are generally considered to be the most efficient and versatile (U.S. Army, Corps of Engineers, 1983). The cutterhead dredge has a rotating cutter around the intake end of the suction pipe and can dig and pump all types of alluvial materials and compacted deposits such as clay and hardpan. Suction pipe diameter ranges from 8 to 30 in.

Cutterhead dredges consist generally of a cutter, ladder, suction pipe, A-frame, H-frame, pumps, spud frame and spuds, and auxiliary equipment. The ladder carries the cutter, suction pipe, lubrication lines, and usually the cutter motor. Dredge ladders are from 25 to 225 ft in length, and the length of ladder determines maximum dredging depth. Dredging may be done to depths of 150 ft with standard ladders in light silty materials. The dredge is held in position or moved ahead with spuds, and the dredge operates by swinging about one spud with the head describing an arc, Figure 10.13d. As the swing is completed, the second spud is lowered, and the other spud raised to make a swing in the opposite direction, and the dredge advances forward.

For open-water disposal, only a floating discharge line is needed with a cutterhead dredge.. The floating discharge line is made up of sections of pipe from 30 to 50 ft long, each supported by pontoons. If land disposal is used, additional sections of shore pipe, usually 10 to 15 ft long, are also needed, Figure 10.14.

Dustpan dredges are self-propelled vessels designed for working in noncohesive material in rivers or sheltered waters with no significant wave action, Figure 10.15. Dustpan dredges have a wide, flared, flat mouth up to 30 ft across on a rigid ladder, and the dredge head is equipped with pressure water jets that loosen the bottom material and suction openings through which the dredged material and water are drawn into the suction line as the dredge is winched forward. Dustpan dredges cut a channel the width of the head and are limited to making relatively shallow cuts in repetitive passes over the shoaled area. They normally discharge into open water through a relatively short pipeline up to 1000 ft long; a longer disposal line requires a booster pump. They can readily be moved outside the navigation channel to let traffic pass.

10.7 Dredged Material Disposal

The Corps of Engineers has been involved in improving channels for navigation since 1824, and the first major program for increasing navigable depth by dredging was authorized in 1896 to provide a 9-ft channel from Cairo, Illinois, to the Gulf of Mexico. For many years the material removed in dredging operations was considered a waste material except when used as fill for commercial or industrial development or to fill in dike fields and old bendways in rivers. However, in recent years, the environmental effects of dredged material disposal has become highly suspect in the public view, and much controversy has ensued.

The major problems associated with disposal of dredged material are:

a. Ensuring availability of sufficient disposal area for initial and future maintenance dredging within a reasonable (economically feasible) distance of dredging operations.

b. Potential adverse environmental effects associated with disposal of dredged material, including increase in turbidity, resuspension of contaminated sediments, and decrease in dissolved oxygen.

Disposal of dredged material usually takes place in one of the following areas:

a. Open water.

b. Elsewhere in the river cross section, as in deep troughs in bends that greatly exceed required navigable depth, in old river bends that have been cut off, and in dike fields or landward of other rectification structures.

c. Dry land in diked disposal areas.

d. Marsh or wetland areas near the river, either with or without retention dikes.

There is increasing interest in the use of dredged material as a resource in the United States because the amount of material dredged each year continues to increase and increasing urbanization and industrial development near waterways and ports has made it difficult to locate new sites for dredged material disposal in many areas. Environmental regulations also have restricted both land and water disposal options. The cost of dredged material disposal has increased rapidly in recent years with greater distances from the dredging site to the disposal area and with environmental controls. Potential environmental impacts can be minimized by using the most suitable dredge type and dredge size and by careful monitoring and control of dredging and disposal operations.

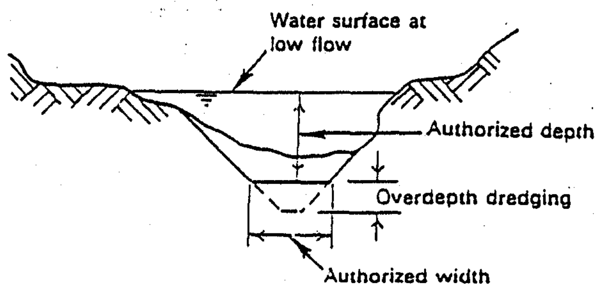


Figure 10.1. Authorized channel dimensions and overdepth dredging for advance maintenance.

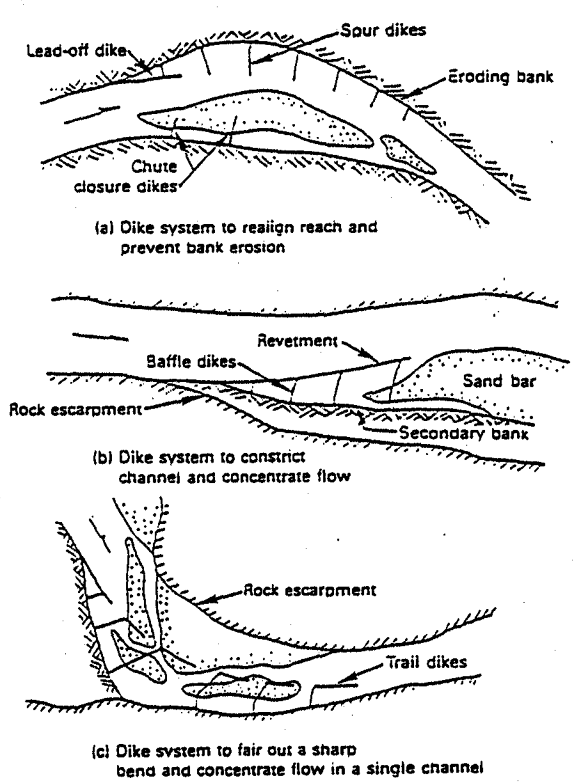


Figure10.2. Dike systems, Arkansas River Navigation Project

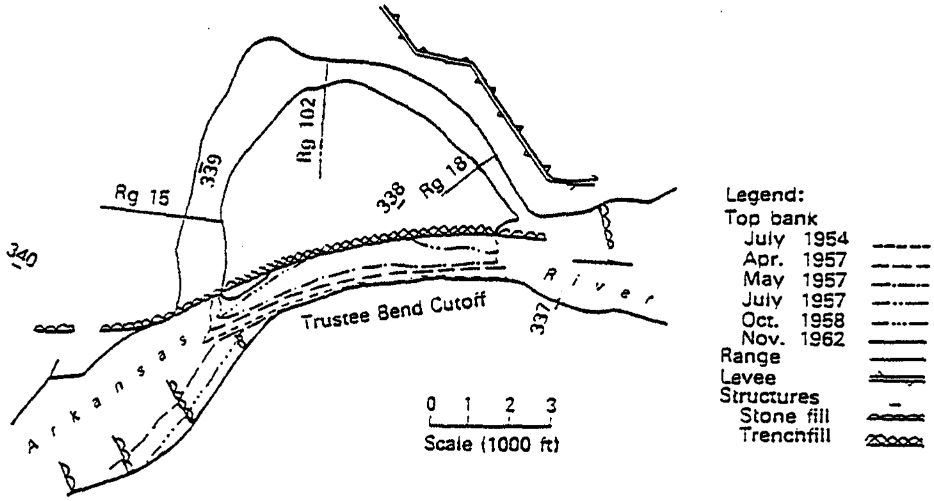


Figure 10.3. Trustee Bend Cutoff, Arkansas River Navigation Project (Corps of Engineers).

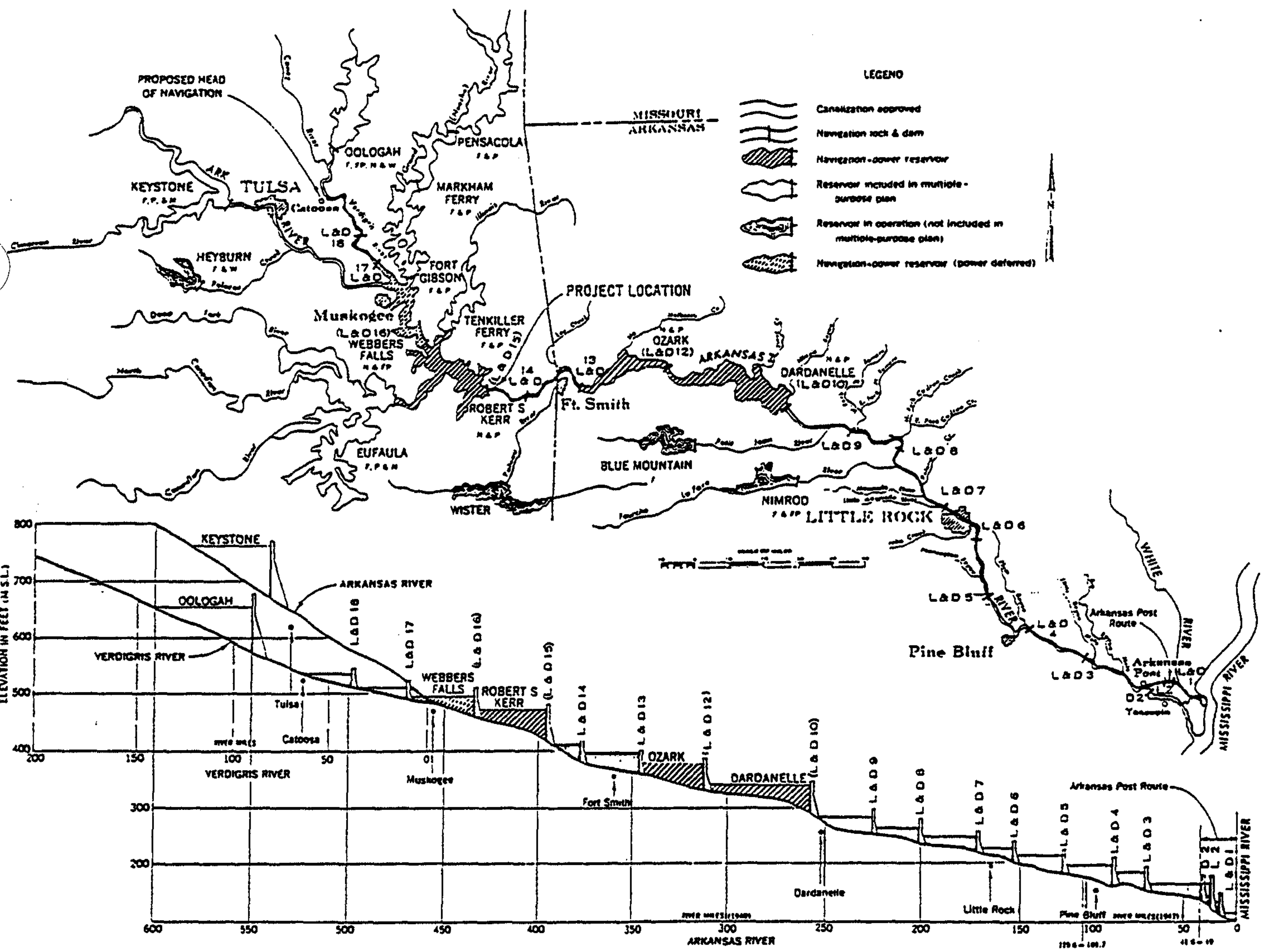


Figure 10.4. Arkansas River Navigation Project (Corps of Engineers).

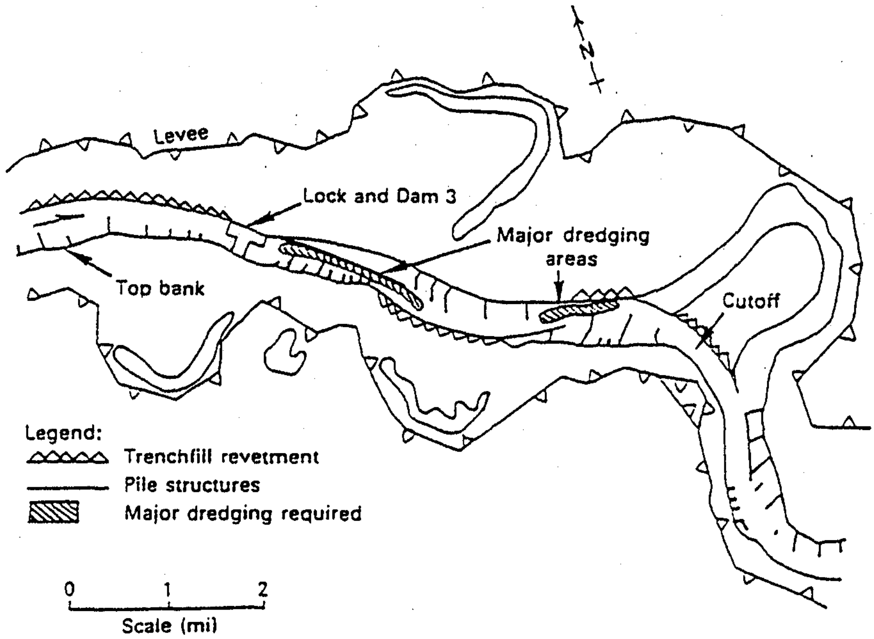


Figure 10.5. Major maintenance dredging reaches at head of Pool 2, Arkansas River Navigation Project (Schmidgall, 1972).

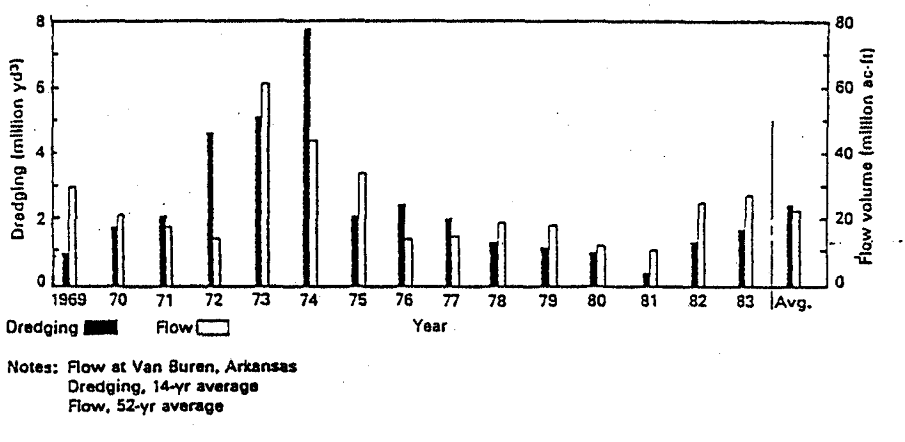
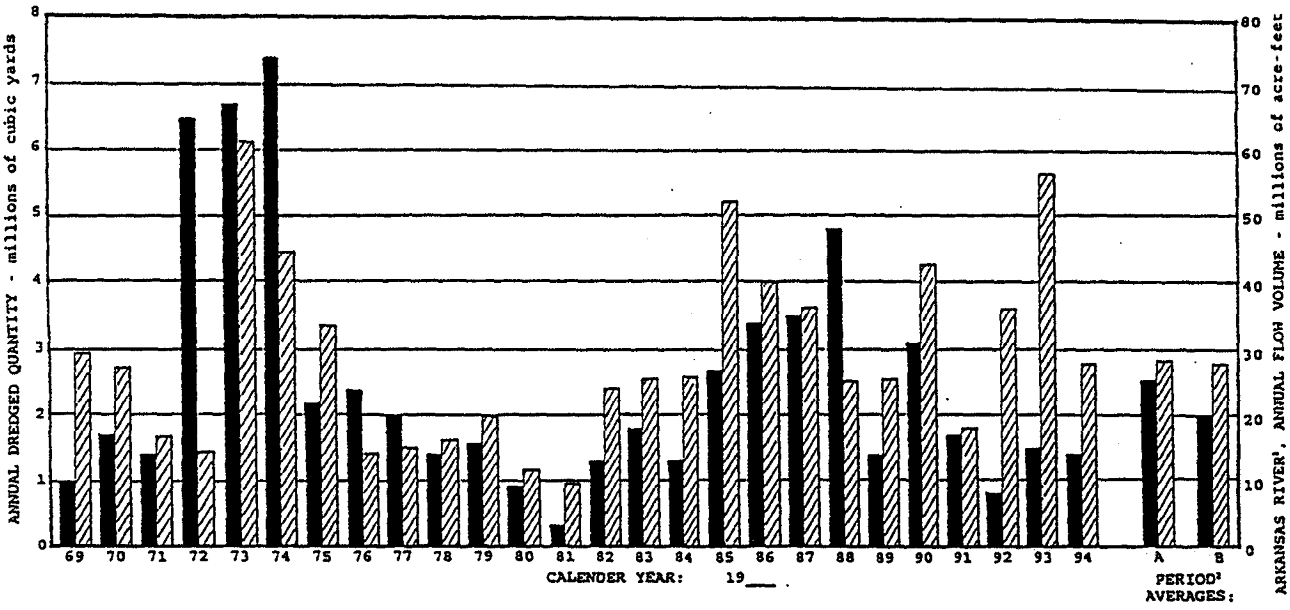


Figure 10.6a. Annual maintenance dredging in Arkansas and flow at Van Buren gage, Arkansas River Navigation Project (Schmidgall, 1981).



Dredging ■ Flow ▨

Dredging and flow averages
Period A. from 1969 through 1994.
Period B. from 1975 through 1994.

Figure 10.6b. Annual maintenance dredging in McClellan-Kerr Navigation System and flow at Van Buren gage (Schmidgall, 1995).

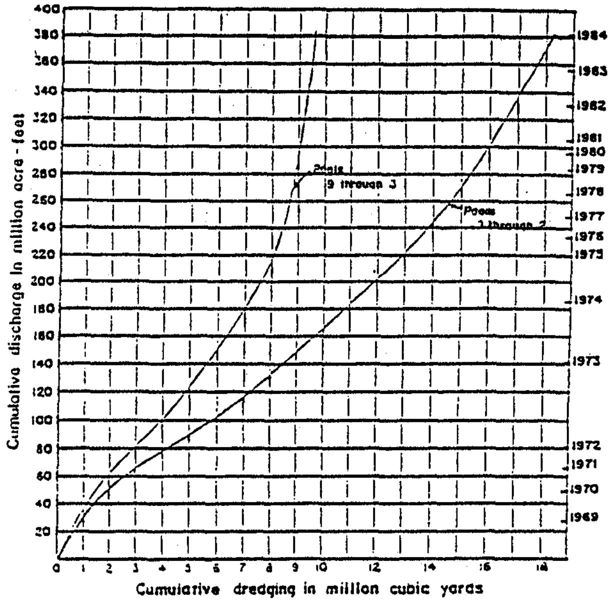


Figure 10.7. Cumulative dredging as a function of discharge, Arkansas River Navigation Project (Petersen and Laursen, 1986).

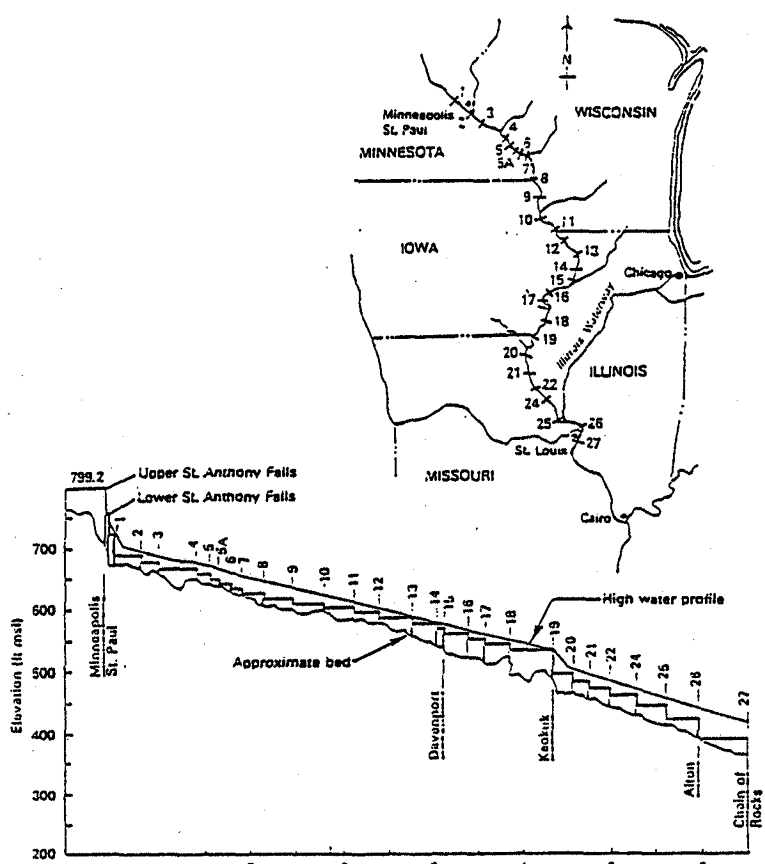


Figure 10.8. Upper Mississippi River Canalization Project (Corps of Engineers).

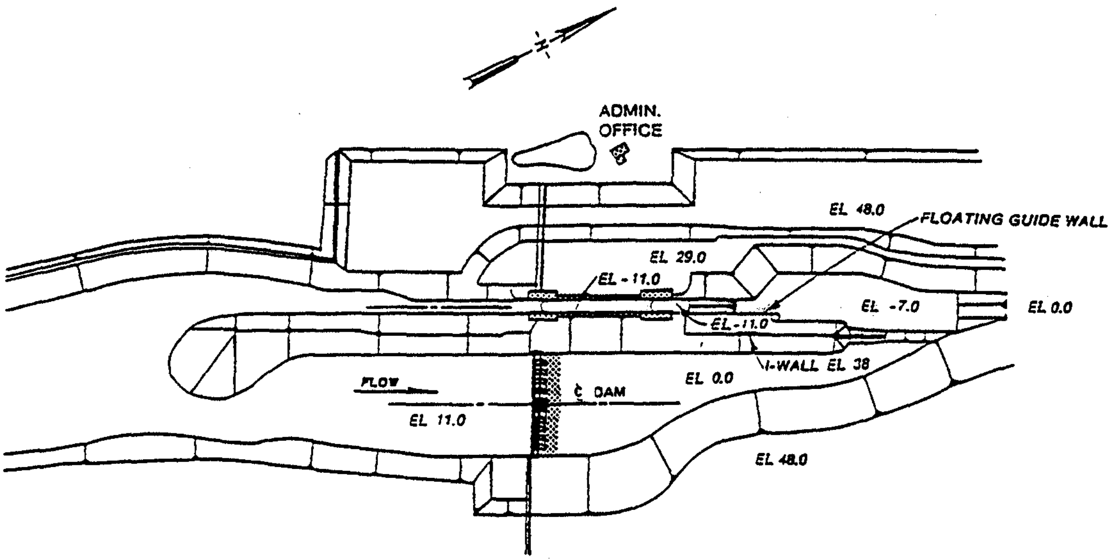


Figure 10.9. Lock and Dam 1, Red River Navigation Project, as constructed (Corps of Engineers).

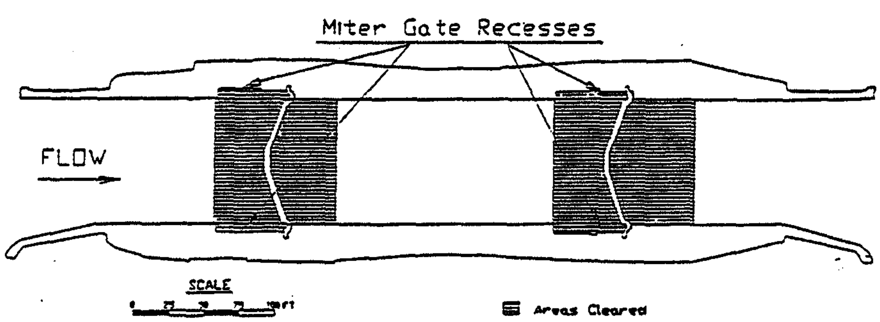


Figure 10.10. Areas cleared of sediment with submersible pump, typical lock, Red River Navigation Project (Neilans, et al., 1993).

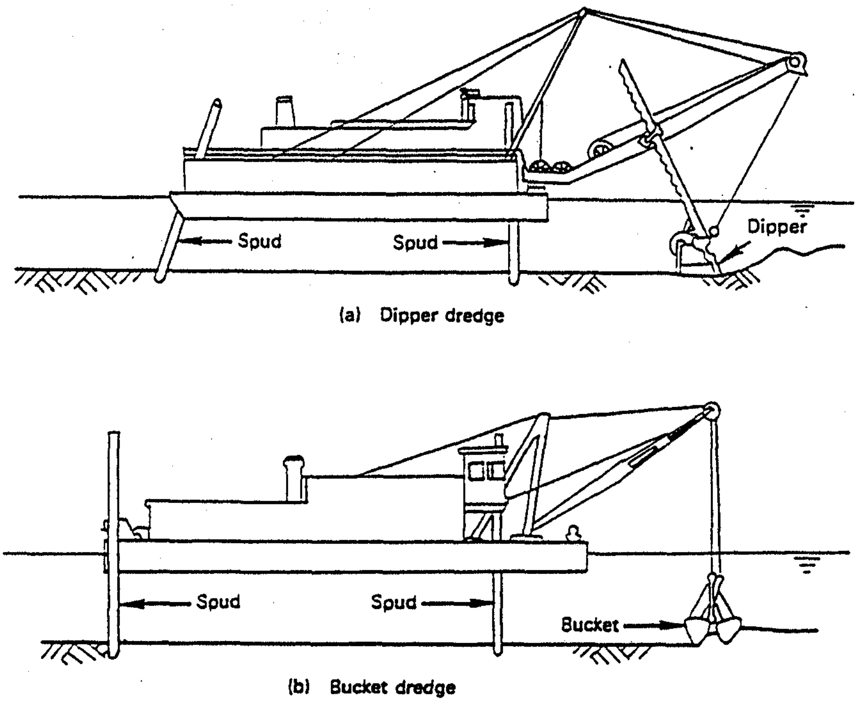


Figure 10.11. Types of mechanical dredges (U.S. Army, Corps of Engineers, 1983).

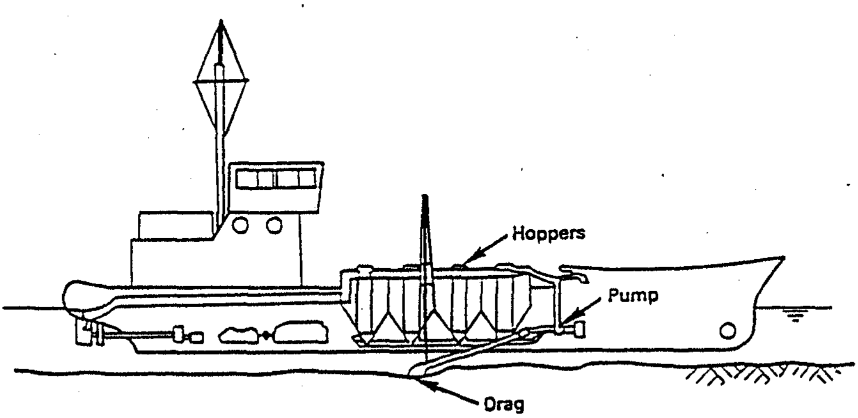
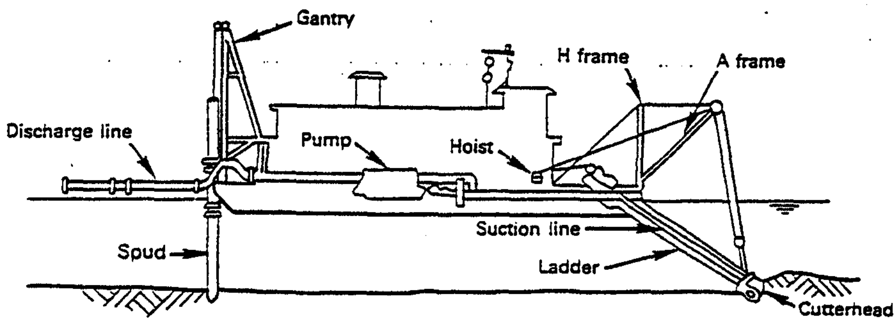
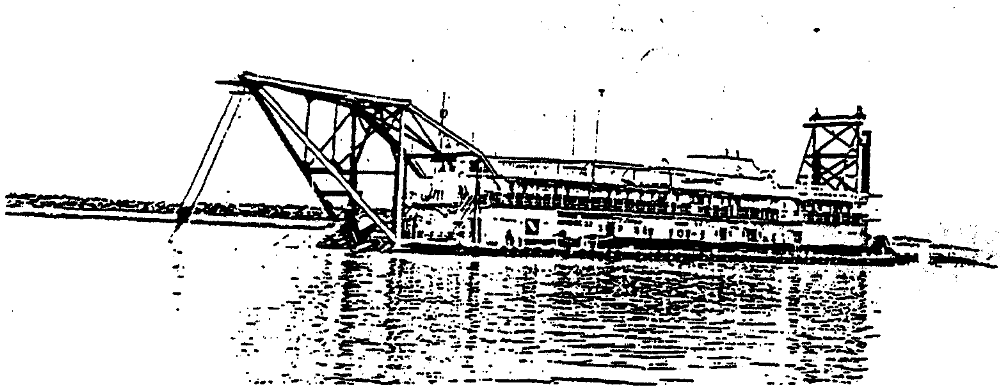


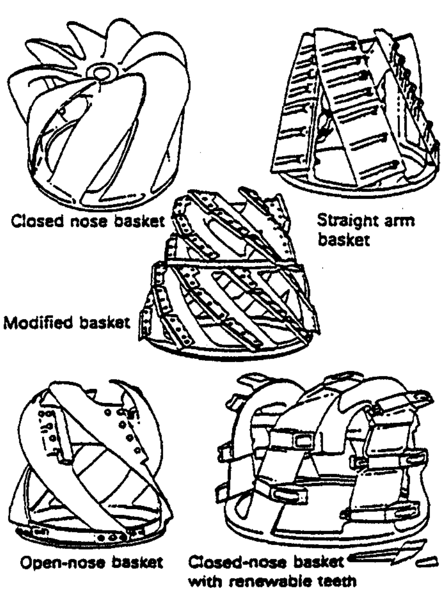
Figure 10.12. Self-propelled seagoing hopper dredge (U.S. Army, Corps of Engineers, 1983).



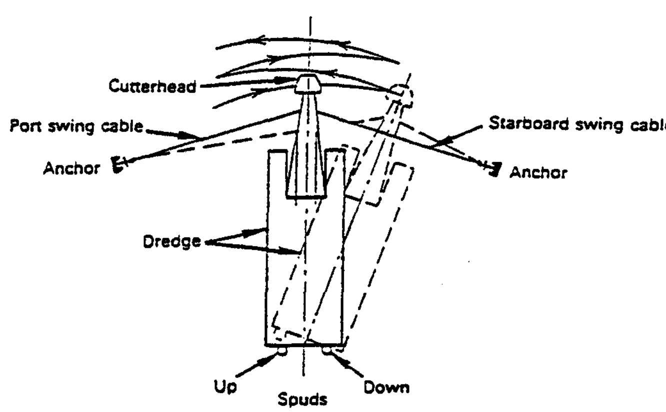
a. Hydraulic pipeline cutterhead dredge.



b. Cutterhead dredge 32, Bauer Dredging Co., with ladder submerged.



c. Types of cutterheads.



d. Operation of a cutterhead dredge viewed from above.

Figure 10.13. Cutterhead dredges (U.S. Army Corps of Engineers).

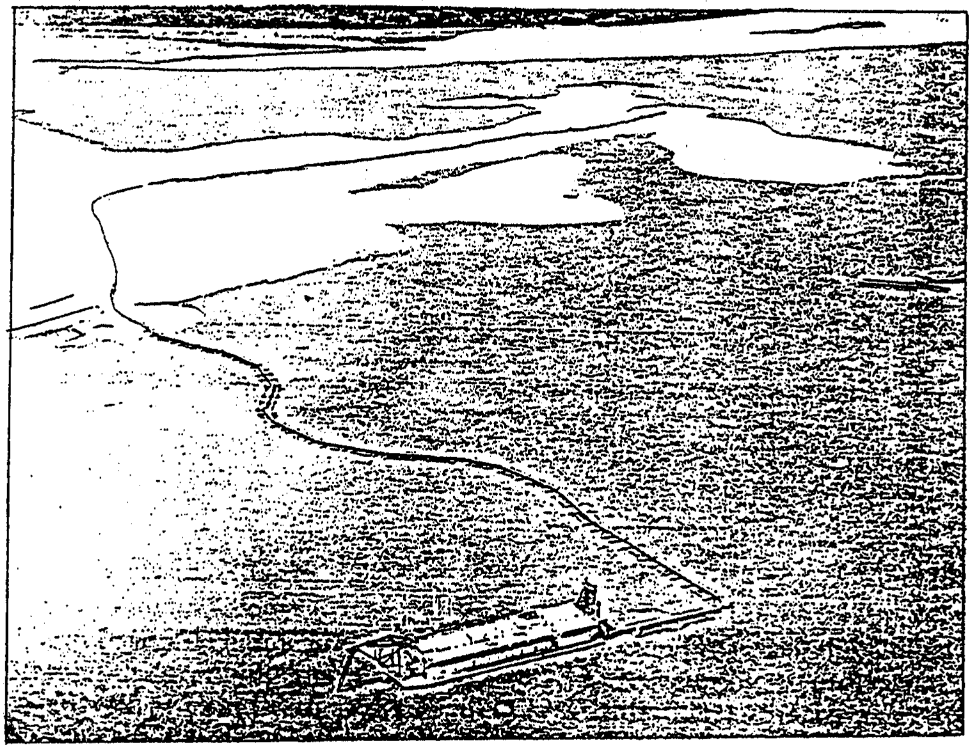


Figure 10.14. Pipeline cutterhead dredge with floating and shore discharge line, Lower Mississippi River
(U.S. Army, Corps of Engineers).

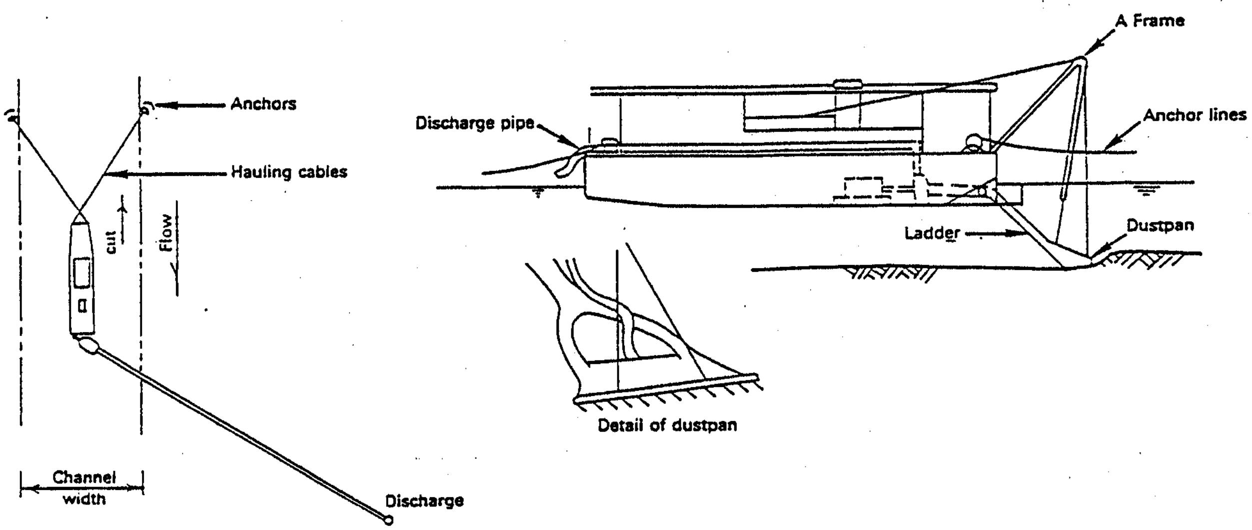


Figure 10.15. Dustpan dredge (U.S. Army, Corps of Engineers).

11. INNOVATIVE LOCK DESIGN

Settlement and development of the interior of the United States was initially by way of the Ohio and Mississippi River, and the earliest navigation developments were on those rivers. The first Federal public works program was clearing and snagging for navigation on the Ohio River in 1842, and the first navigation lock and dam was constructed on the Ohio River, about five miles below Pittsburgh, in 1885. It was very successful and led to construction of other navigation works on the Upper Mississippi, Ohio, and tributaries, Figure 11.1.

By 1929, 981 miles of the Ohio River had been canalized by 52 locks and dams to provide a 9-ft channel. Later the original 52 structures were replaced with 20 higher lift locks and non-navigable dams, Figure 11.2. Also there are 51 locks and dams on Ohio River tributaries. Total length of the Ohio system is 2776 miles. The standard tow has 15 barges (3 wide, 5 long) and a towboat.

By 1940, 850 miles of the Upper Mississippi River had been canalized by 26 locks (110- by 600-ft) and dams. Later larger locks were constructed at some locations; a lateral canal with two locks was constructed near St. Louis; and navigation was extended 4.6 miles upstream at Minneapolis by construction of two smaller locks at St. Anthony Falls. The system now has 29 locks, Figure 11.3. The Upper Mississippi and tributaries now provide 1982 miles of navigable waterway. The standard tow is the same as on the Ohio River.

11.1 Need for Rehabilitation or Replacement of Navigation Structures

Annual waterborne tonnage on the Ohio River increased from 22 million tons in 1930 to 151 million tons in 1982. On the Upper Mississippi, tonnage increased from 3.1 million tons in 1940 to 91 million tons in 1982, Figure 11.4.

Advances in towing equipment over the years, including towboats with increased engine horsepower, have allowed larger and heavier tows to be moved and have extended navigation to periods of relatively high flows and moderate ice conditions.

Modern tows are larger than the tows for which the locks and lock approaches were designed. This, combined with higher entrance velocities at the locks during high flows, has created navigation problems at the older locks. The longer tows require double lockage, and operators must use extreme care in entering the locks. For example, lockage at the small old Gallipolis Locks on the Ohio River required multiple lockages and as much as 4.5 hours to lock through a single tow, while tows pass through the 1200-ft locks on the Ohio in about an hour. Such delays are costly for shippers. In 1992 delays of tows at five of the locks and dams on the Upper Mississippi totaled 87,000 hours, representing an estimated loss to shippers of $35 million.

When navigation projects on the Ohio and Upper Mississippi Rivers were designed and constructed, projections of future traffic were much lower than what has actually occurred. Thus, the locks have been more heavily used than foreseen, and expensive and frequent maintenance has been needed. However, budget constraints have limited maintenance work, and many of the older structures have deteriorated.

There is need for a systematic, effective, and adequately funded maintenance program for waterway systems to remain useful and efficient. While routine maintenance is generally all that is required in the early years of project operation, unforeseen construction may be needed later to enhance operation or correct deficiencies. It is important to perform maintenance as needed, before problems become major and require closure of the waterway for major repairs.

Traffic from both the Upper Mississippi River and the Illinois Waterway passed through the old Lock and Dam 26, on the Upper Mississippi at Alton, The old lock was both inadequate to handle the size and number of tows on the river in the early 1980s and had serious structural problems. The old 110- by 600-ft lock had an estimated maximum lockage capacity of 73 million tons per year. In 1981, 70.3 million tons passed through the old lock (total value of cargo was $14 billion). In 1982, a recession year, 68 million tons passed through, with an average delay of tows of 10 hours due to backup of traffic.

Structural problems at Lock and Dam 26 (Niemi, 1986) included lateral and vertical movement of both the lock and dam which were supported on vertically driven timber and concrete piles. The stilling basin floor (3.5 ft thick) had eroded as much as 2 to 3 ft, and voids were found in the foundation alluvium under the dam and lock guide walls. Major emergency rehabilitation work was undertaken in 1970 and 1971, and other repairs were made annually in later years. Studies indicated it would be less costly to construct a new facility that to rehabilitate the existing structures, and Melvin Price Locks and Dam were recently completed to replace Lock and Dam 26.

The Melvin Price Locks and Dam project is located 2 miles downstream from the old Lock and Dam 26 and includes one 1200-ft and one 600-ft lock. The dam has nine tainter gates (100-ft wide by 42 ft high) and an overflow dike on the west bank. Two of the gate bays are located between the locks. All spillway gates are operated at uniform opening. The 350-ft separation between the locks allows simultaneous approach and departure of tows. Total project cost was in the order of $974 million. Capacity of the new 1200-ft lock is estimated to be 94 to 100 million tons per year; capacity of both locks is estimated to be about 179 million tons per year and is expected to meet needs for the next 50 years.

11.2 Increasing Lock Capacity

Traffic capacity of locks can be increased somewhat by such measures as:

a. Improving the hydraulic system or modifying the emptying system.

b. Improving lock approaches by widening or realignment, and improving the upper approach by installing submerged dikes.

c. Instituting regulations for locking order to shorten lockage time. That is, passing a specified number of tows in one direction, then passing tows in the other direction, rather than locking tows through in order of arrival.

d. Establishing hours for locking recreational boats, or constructing a recreation boat lock.

e. Using helper boats to move unpowered barges.

f. Requiring that large tows have bow thrusters.

Major increase in lock capacity can be realized only by providing additional lock chamber space which can be done by:

a. Lengthening existing locks.
b. Replacing existing locks with larger locks.
c. Constructing additional locks.

11.3 Need for Innovations in Lock Design

The need for additional or replacement locks at many Corps projects becomes more critical each year, and construction costs have escalated dramatically, partly because replacement locks are larger than the structures being replaced. The cost of a 600-ft lock and dam on the Arkansas River in the 1960s was in the order of $10 million. A similar lock and dam on the Red River, completed in December 1994, cost about $115 million.

Total cost of the recently completed Melvin Price Locks and Dam on the Mississippi was about $970 million, but that project included one 1200- and one 600-ft lock and more spillway capacity than on the Arkansas or Red Rivers. Estimated cost of the Olmsted project now under construction on the lower Ohio River, which includes two 1200-ft locks, is in the order of $1.2 billion.

Navigation projects in the United States formerly were Federally funded. However, the Waterways Development Act of 1986 requires that funding for new locks and for major rehabilitation work be shared 50-50 by the Federal government and the Inland Waterways Trust Fund. The Trust Fund derives its monies from a tax on fuel used on the inland waterways system, currently 20 cents per gallon, and such revenues are limited. This restricts the number of replacement and rehabilitation projects that can be undertaken each year, and the backlog of critically needed work increases each year. Accordingly, the Corps of Engineers for the past few years has vigorously pursued a program seeking innovative and less costly designs to restore the aging navigation infrastructure.

11.4 Innovative Lock Design Program

The primary focus of the Corps' innovative lock design effort involves replacing conventional gravity lock chamber walls with less costly thin walls between the miter gate monoliths. At existing locks, filling and emptying culverts are located in the lock walls for all the commonly-used filling and emptying systems. The new thin-wall concept would locate the filling and emptying culverts on the floor of the lock chamber, and the intake systems could be placed in the upper miter gate sill, Figure 11.5. Vertically mounted butterfly valves are proposed for use as culvert control valves.

Because these new concepts are very different from conventional designs, the Corps has undertaken a series of model studies to investigate hydraulic performance of the new filling and emptying systems to ensure they will perform acceptably.

A model testing program has been set up at the USACE Waterways Experiment Station

(WES), Vicksburg, Mississippi, to investigate suitability of the new concepts for design of filling and emptying for new locks proposed at four sites: McAlpine Lock and Dam, Ohio River; Marmet Lock and Dam, Kanawha River; Monongahela River No. 4 Lock and Dam; and a representative lock on the Upper Mississippi.

Other new concepts involve modification of the upstream guide and guard wall designs. However, the greatest savings in construction costs comes from placing the filling and emptying system on the lock chamber floor instead of in the lock walls. About 15 percent less material would be needed for the thin wall design, but the most significant savings would be in placing concrete for the walls without having to form for the culverts and in reinforcing steel. It is likely that a roller-compacted concrete base with a cast-in-place cap and lock face could be used for the walls, Figure 11.6. Also, the wall foundation can be higher, cutting down on rock excavation. Such modifications are expected to lessen the construction period significantly.

Winfield Locks and Dam, Kanawha River. Some of the innovative design concepts are included in new twin 110- by 800-ft locks now under construction at Winfield, on the Kanawha River. The old twin locks at Winfield are the busiest locks on the inland waterways system, with over 20,000 lockages per year. The existing locks are 56- by 360-ft and can accommodate only one modern jumbo barge of the type used to transport coal in the region. Typical coal tows are composed of five barges that must be locked through one at a time at Winfield, requiring about 3.5 hours for a single tow to pass through. Under adverse conditions, as long as five hours is required, and tows often wait 24 hours before being locked through. These delays represent a loss to shippers of about $17 million annually. The new 800-ft locks will be capable of passing a 9-barge tow in a single lockage.

The upstream guide wall along the shore will have wide-flange steel piles grouted into rock, with a reinforced concrete cap and skirt, instead of a continuous sheetpile wall. The length of the wall will be 1000 ft, about half the usual length, and the remaining length will have the bank sloped back and riprapped.

The upstream guard wall will have half as many concrete-filled sheet pile cells as normally used, doubling the opening between cells to about 105 ft. There will be no pile arcs between the cells, and post-tensioned cap beams will be used, rather than reinforced concrete.

These modifications of the upstream approach walls are estimated to have reduced the cost of that work by more than one-third, or by about $5 million.

The first stage contract for construction of the cofferdam was completed in 1991. The contract for construction of the new lock and a 100-ft wide spillway bay between the old and new locks was awarded in May 1994. Pouring concrete began in April, 1995, and the lock is scheduled to begin operation in spring, 1997.

Modeling program. The modeling program currently underway at WES is set up in two phases to make the best use of available time, facilities, and manpower. Phase 1 testing began in March 1995. Model components and a lock facility needed for Phase 2 testing were completed in late 1995, and testing has been initiated.

- Phase 1 involves testing intake models to investigate site-specific intake and approach conditions since this is likely to be one of the most difficult design features of the new filling and emptying design.

- Phase 2 involves testing the proposed filling and emptying designs and the lock outlets.

Intake models for McAlpine, Marmet, and Monongahela No. 4 Locks will be used to identify any undesirable flow patterns in the approach areas to the locks, such as strong vortices or concentrated flows, and to refine the intake designs if improvements are needed. Intakes located in the miter gate sill are particularly susceptible to these types of flow conditions. In addition, the performance of the proposed intake and trashrack will be investigated, and velocities in the intake area will be measured to help evaluate effects on tows in the area.

Testing of two filling and emptying models began in the summer of 1995. The first model was used to develop a filling and emptying system for McAlpine, Marmet, and Monongahela No. 4 since proposed designs and project features are similar for these three locks. The second filling and emptying model will be for the Upper Mississippi lock. Model testing of the filling and emptying systems will include determining optimum location of culvert ports and the need for baffles to deflect jets from the ports and reduce hawser stresses; evaluating flow distribution in the lock chamber with free tow drift patterns, measurement of longitudinal and transverse hawser forces, and observation of surface currents. Different valve operations and associated filling and emptying times will be tested, and average pressure measurements will be obtained throughout the system. Performance of the lock outlets will be studied.

Tests initiated on the McAlpine intake model in March 1995 are complete. Preliminary results indicate that it may be desirable to use extensions on ports in the laterals to better direct jets issuing from the ports perpendicularly to the culverts; to use baffles along the walls and centerline of the lock to redirect the jets and reduce hawser stresses; and to relocate the intake ports from the miter gate sill to the approach walls (with externally mounted butterfly valves) to reduce vortex problems.

Testing of the Marmet intake model and the McAlpine filling and emptying system was scheduled to be completed in late 1995.

Marmet Lock and Dam, Kanawha River. Marmet Lock and Dam is next upstream from the Winfield project. The same upstream approach wall modifications adopted for Winfield will be used at Marmet. In addition, the new filling and emptying system concept with culverts on the lock floor is expected to result in significant savings. It is also proposed to eliminate traditional vertical lift gates for emergency closure and the low-sill passage for use by emergency craft if the upper pool is lost. This will permit the upper gate sill to be raised about 28 ft, reducing cost of the miter gates and the cost of dredging in the upstream lock approach.

It is estimated that the new Marmet Lock would cost in the order of $243 million if traditional design criteria are used. It is expected this cost can be reduced by about $50 million (20 percent) if innovative design concepts are adopted.

11-5 Other Innovative Concepts

Schmidgall (1995) has suggested two other areas were innovative design could enhance lock operation:

a. Model tests and prototype operation of locks with elaborate bottom longitudinal filling and emptying systems have shown such locks can be operated satisfactorily with valve opening times of one minute. Schmidgall suggests that lock filling time could be shortened and the low pressure problems downstream of the filling valve at high-lift locks could be minimized if the valve opening time were reduced to 15 or 10 seconds. With such a fast operating speed, the valves would quickly pass through the partial gate opening settings that create negative pressures before flow momentum has stabilized sufficiently to cause the low pressure problems.

b. An improved tow haulage system could significantly shorten the time required for double lockages. Most systems now in use were not installed at the time the locks were constructed, but were added later. These systems, which are typically cable, pulley, and winch systems are located on top of lock walls and interfere with miter gate operation. Double lockage times could be significantly reduced if the unpowered half of a tow could be pulled out of the lock chamber and secured beyond the lock long enough for the powered half to lock through and reattach to the unpowered half.

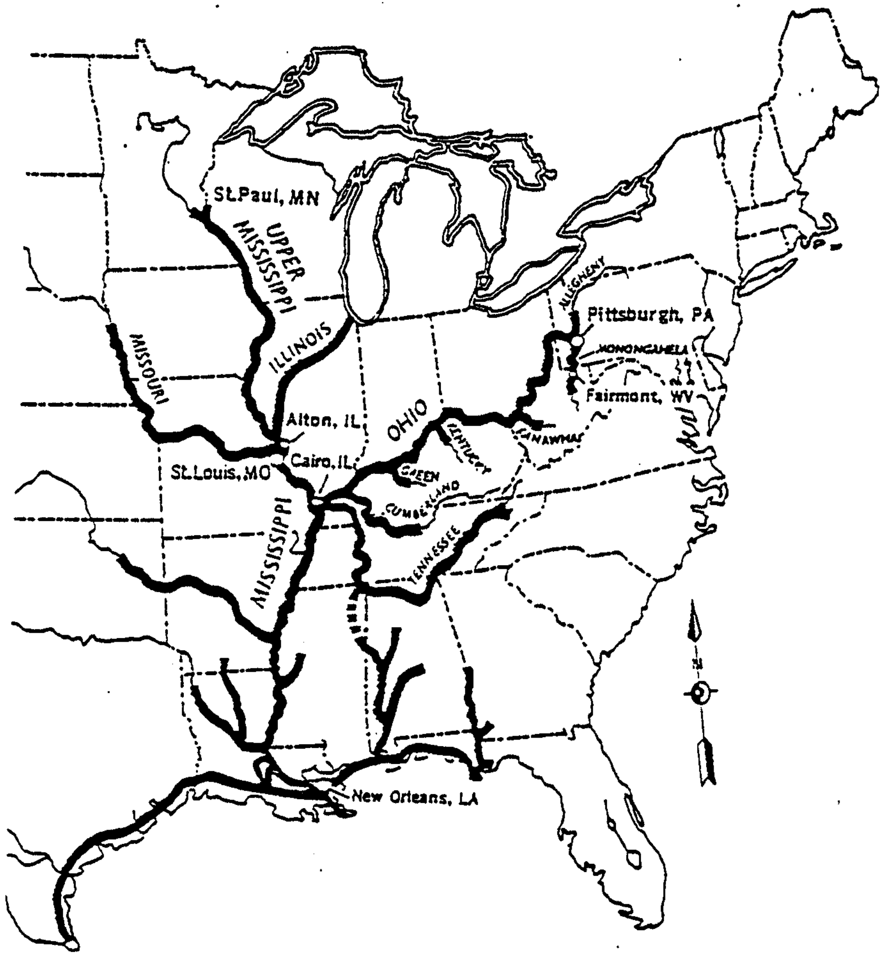


Figure 11.1, Upper Mississippi and Ohio River Navigation Systems and Connecting Waterways (Corps of Engineers).

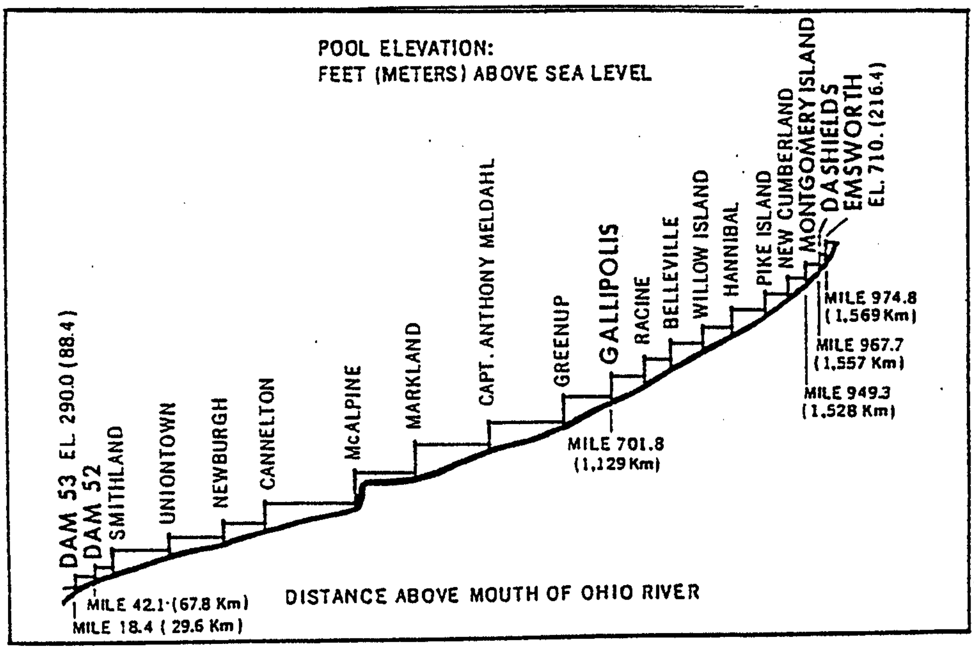


Figure 11.2, Profile of Ohio River Navigation Pools (Corps of Engineers).

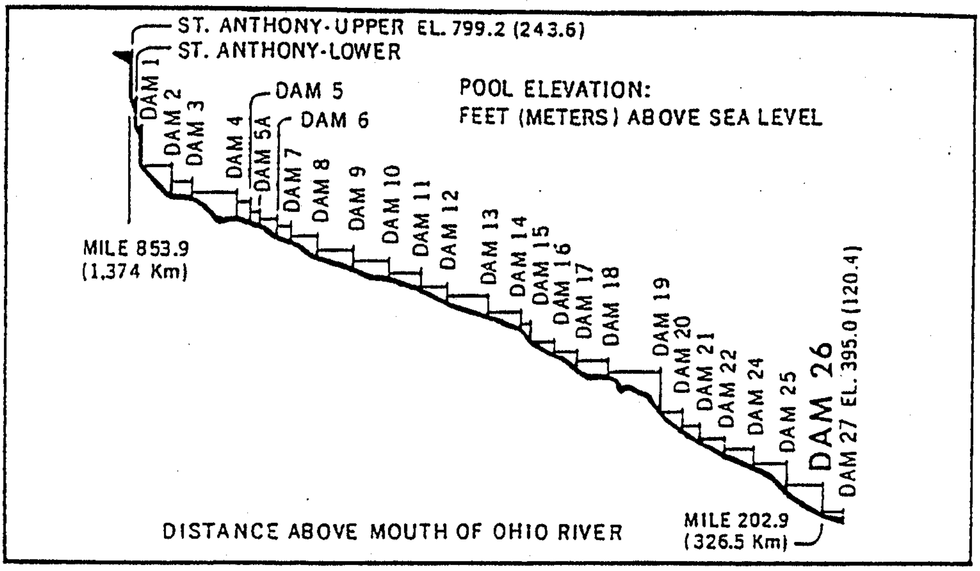


Figure 11.3, Profile of Upper Mississippi Navigation Pools (Corps of Engineers).

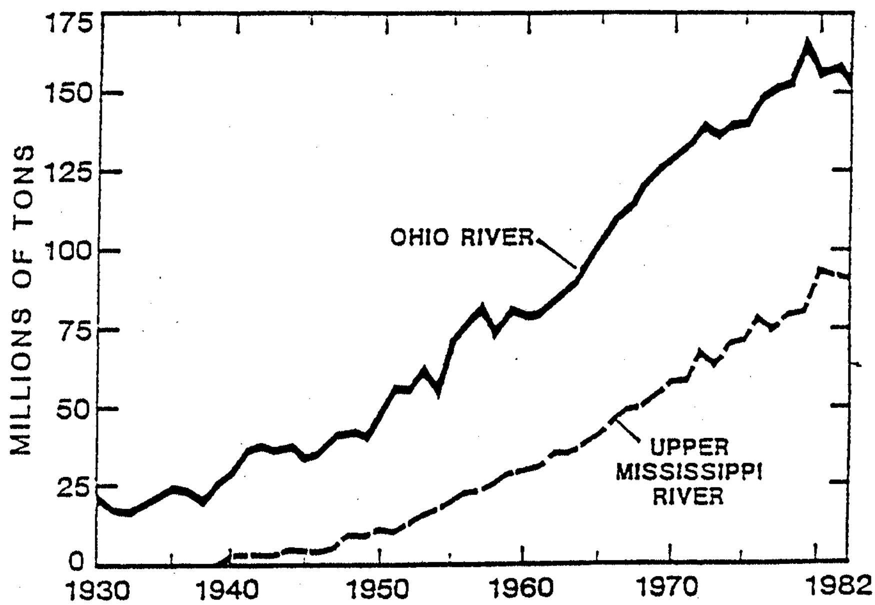


Figure 11.4, Growth of Waterborne Commerce,
Ohio and Upper Mississippi Rivers
1930 - 1982.
(Corps of Engineers)

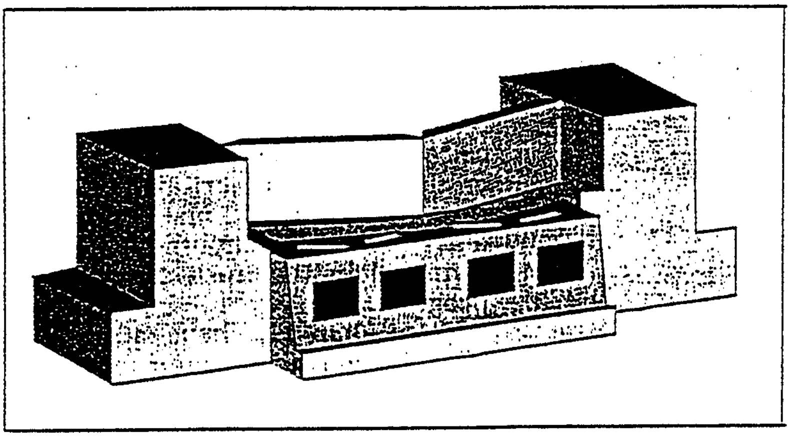


Figure 11.5, Intakes in Upper Miter Gate Sill.

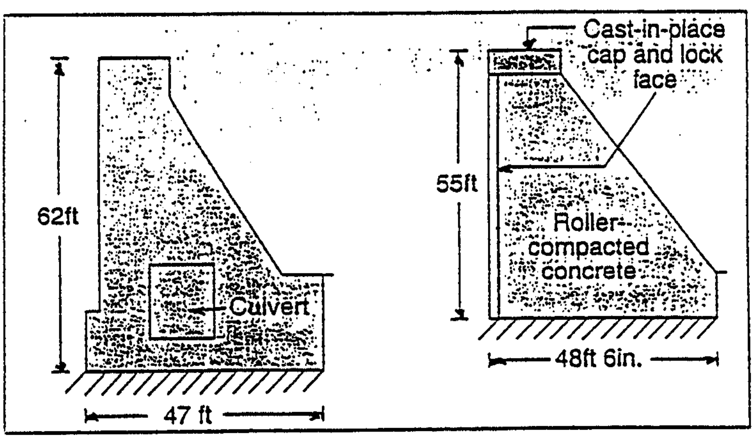


Figure 11.6, Lock Wall Monoliths with Culverts in Lock Wall and on Floor of Lock Chamber.

APPENDIX A. REFERENCES

Ables, J. H., Jr., *Filling and Emptying System for Bay Springs Lock, Tennessee-Tombigbee Waterway, Mississippi,* U.S. Army, Corps of Engineers, Waterways Experiment Station, Technical Report H-78-19, 1978.

Ables, J. H., Jr., and M. B. Boyd, *Filling and Emptying System, Dardanelle Lock, Arkansas River*, U.S. Army, Corps of Engineers, Waterways Experiment Station, Technical Report H-69-5, 1969.

Antle, L. G. and D. V. Grier, Prioritizing Inland Waterway Infrastructure Needs, *in Bulletin No. 87, Permanent International Association of Navigation Congresses,* 1995.

Cooper, D. R., *Submersible-type Tainter Gate for Spillway, Marseilles Lock and Dam,* U.S. Army, Corps of Engineers, Waterways Experiment Station, Technical Report HL-89-1, 1989.

Combs, P., and W. H. Espey, Jr., Sediment Management Challenges of the Red River Waterway Project, *ASCE Proceedings of the National Conference on Hydraulic Engineering,* H. H. Chang and J. C. Hill, eds., 1990.

Copeland, R. R., and W. A. Thomas, *Red River Waterway Sedimentation Study Downstream from Lock and Dam 1,* U.S. Army, Corps of Engineers, Technical Report HL-88-15, 1988.

Davidson, R. A., *Gallipolis Lock Intake Vortex Study,* Ohio River, U.S. Army, Corps of Engineers, Technical Report 87-17, 1987.

Davis, J. P., *Hydraulic Design of Navigation Locks,* U.S. Army, Corps of Engineers, Waterways Experiment Station, Miscellaneous Paper HL-89-5, 1989.

Derrick, D. L., *Shallow-Draft Training Structure Current Repair Practices and Repair Guidelines,* U.S. Army, Corps of Engineers, Waterways Experiment Station, Technical Report REMR-HY-8, 1991.

Elder, R. A., J. T. Price, and W. W. Engle, Navigation Locks: TVA's Multiport Lock Filling and Emptying System, *ASCE Journal of the Waterways and Harbors Division,* February, 1964.

Franco, J. J., *Development of Navigation with Locks and Dams.* U.S. Army, Corps of Engineers, Waterways Experiment Station, Miscellaneous Paper H-76-15, 1976.

Franco, J. J., and J. E. Glover, *Navigation Conditions at Robert S. Kerr Lock and Dam,* Arkansas River, Waterways Experiment Station, Technical Report H-68-5, 1968.

Franco, J. J., and C. D. McKeller, Jr., *Navigation Conditions at Ozark Lock and Dam, Arkansas River,* U.S. Army Corps of Engineers, Waterways Experiment Station, Technical Report H-68-10.

Grace, J. L., Jr., *Spillway for Typical Low-Head Navigation* Dam, *Arkansas River, Arkansas,* U.S. Army, Corps of Engineers, Waterways Experiment Station, Technical Report 2-655, 1964.

Huval, C. J., *Lock Approach Canal Surge and Tow Squat at Lock and Dam 17, Arkansas River Project,* U.S. Army, Corps of Engineers, Waterways Experiment Station, Technical Report HL-80--17.

Little, C. D., Jr., Numerical Model Analysis of Sedimentation Problems at Lock and Dam 1, Red River Waterway, *ASCE Proceedings of the National Conference on Hydraulic Engineering,* R. M. Ragan, ed., 1987.

Maynord, S. T., *Safe Navigation Speeds and Clearance at Lower Sill, Temporary Lock 52, Ohio River,* U.S. Army, Corps of Engineers, Waterways Experiment Station, Technical Report 87-3, 1987.

Murphy, T. E., *Lock Design, Sidewall Port Filling and Emptying System.* U.S. Army, Corps of Engineers, Waterways Experiment Station, Miscellaneous Paper 4-75-7, 1975.
Murphy, T. E., *Lock Design, Bottom Longitudinal Filling and Emptying System.* U.S. Army, Corps of Engineers, Waterways Experiment Station, Miscellaneous Paper HL-80-5, 1980.
Neilans, P., J. Clausner, J. Coldiron, and C. Corkern, Maintenance applications of a submersible pump on the Red River Waterway, in U.S. Army, Corps of Engineers. Waterwaus Experiment Station, *Dredging Research*, Vol DRP 93-2, May, 1993.
Nelson, M. E., and H. J. Johnson, Navigation Locks: Filling and Emptying Systems for Locks, *ASCE Journal of the Waterways and Harbors Division*, February 1964.
Niemi, J. R., *Aging Navigation Structures on the Mississippi and Ohio River Navigation Systems*, 8th Annual USCOLD Lecture Series, Navigation Locks and Dams, New and Old, 1986.
Petersen, M. S., *River Engineering*, Prentice Hall, 1986.
Petersen, M. S., and E. M. Laursen, *Evaluation of Arkansas River Design Criteria for Stabilization and Rectification of the Channel*, Contract No. DACW39-86-C-0027 U.S. Army, Corps of Engineers, Waterways Experiment Station, 1986.
Pinkard, C. F., Red River Waterway: A Sedimentation Challenge, *ASCE Proceedings on the First International Conference on Water Resources Engineering*, W. H. Espey, Jr. and P. G. Combs, eds., 1995a.
Pinkard, C. F., Red River Waterway Project: General Design, *ASCE Proceedings of the First International Conference on Water Resources Engineering*, W. H. Espey, Jr. and P. G. Combs, eds., 1995b.
Robertson, R.R., Jr., Locks and Dams - Red River Waterway, *ASCE Proceedings of the First International Conference on Water Resources Engineering*, W. H. Espey, Jr. and P. G. Combs, eds., 1995.
Schmidgall, T., Maintenance Dredging on the Arkansas River, in *Sedimentation*, H. W. Shen, ed., 1972.
Schmidgall, T., *McClellan-Kerr Arkansas River Navigation System, An Engineering Evaluation,* 1 June 1981 (unpublished).
Schmidgall, T., Twenty-six Years of Dredging on the Arkansas River, 14 December 1995 (unpublished).
Shows, L. L., and J. J. Franco, *Navigation Conditions at John H. Overton Lock and Dam, Red River*, U.S. Army, Corps of Engineers, Waterways Experiment Stations, Technical Report HL 79-3,1979
Stockstill, R. L., *Olmsted Lock Outlet, Ohio River*, U.S. Army, Corps of Engineers, Waterways Experiment Station Technical Report HL 92-13, 1992.
Tate, C. H., Jr., *Bay Springs Canal Surge Study, Tennessee-Tombig*bee Waterway, Mississippi and Alabama, U.S. Army, Corps of Engineers, Technical Report H-78-9, 1978.
U.S. Army, Corps of Engineers, Engineer Manual 1110-2-2606, *Navigation Dams*, 1952.
U.S. Army, Corps of Engineers, Engineer Manual 1110-2-1604, *Hydraulic Design, Navigation Locks, 1956.*
U.S. Army, Corps of Engineers, Waterways Experiment Station, *Dardanelle Lock and Dam, Arkansas River*, Technical Report 2-558, 1960.
U.S. Army, Corps of Engineers, Engineer Manual 1110-2-1611, *Layout and Design of Shallow Draft Waterways*, 1980.
U.S. Army, Corps of Engineers, Engineer Manual 110-2-5025, *Dredging and Dredged Material Disposal*, 1983.

U.S. Army, Corps of Engineers, Engineer Manual 110-2-1605, *Hydraulic Design of Navigation Dams*, 1987.

APPENDIX B. RED RIVER WATERWAY, LOUISIANA

B.1 Project Description

The Red River Waterway was authorized by the U.S. Congress in 1968 with the primary purpose of providing a 9-ft deep by 200-ft wide navigation channel from the Mississippi River upstream to Shreveport, Louisiana, Figure B.1. The project includes five locks and dams, with a total lift of 141 ft. Construction of the project proceded in an upstream direction. Lock and Dam 1 was completed in the fall of 1984; Lock and Dam 2 in the fall of 1987; Lock and Dam 3 in December, 1991; Locks and Dams 4 and 5 in December, 1994. The project includes channel realignment and bank stabilization. Total project cost was approximately $1.8 billion, with about half that cost being for the five locks and dams.

Preproject river length was about 280 miles, and this was shortened about 50 miles, or 18 percent, by realignment work. Shortening has lowered flood profiles. During the May, 1990, high water on the Red River, a flood exceeding the 100-yr frequency event, peak stages at Shreveport were in the order of one to two ft below what would have occurred prior to realignment of the channel (Pinkard, 1995b).

All dam spillways have tainter gates 60-ft long for normal operation to maintain the pool during low-water periods and to pass flood flows. Lock and Dam 1 was designed to pass the design flood (the 100-yr recurrence frequency event) with one ft of swellhead, but 11 gate bays were required. Dams 2 through 5 have tainter gates and an uncontrolled ogee bay with crest set at 2 ft above upper pool level and crest lengths ranging from 150 to 315 ft. For Dams 2 through 5, economic analysis of the costs of additional gates vs the costs of fewer gates plus the costs of flowage easements for inundating additional lands indicated that it would be cheaper to use five or six tainter gate bays and an uncontrolled crest, and obtain additional flowage easements rather than to provide a larger number of tainter gates to pass the design flood with one foot of swellhead, as discussed below.

Stilling basins were designed to provide submergence of 85 percent of the conjugate depth of the entering flow and consist of a concrete slab with two rows of baffle blocks and a sloping end sill (Robertson, 1995).

All locks are 84 ft wide by 800 ft long and have sidewall port filling and emptying systems designed to limit hawser forces to less than 5 ton. Locks are sized for a design tow consisting of six barges (each 35 ft by 195 ft) and a tug. Lock lifts range from 35 ft to 24 ft, and 6-barge tows can pass through a lock in a single lockage in about 25 minutes.

On the Red River, channel velocities become too high for commercial navigation when flow is greater than the 10-year frequency flood (125,000 and 145,000 cfs at Shreveport and Alexandria, respectively). Therefore the top of lock chamber walls was set at an elevation of at least 2 ft above the 10-yr flow line (in the order of 8 ft above normal lower pool level) so that the locks are operational up to the 10-year flow frequency event. During high water, mean channel velocities are about 7 ft/sec and maximum velocities are in excess of 10 ft/sec.

Each approach at Lock 1 has a floating guide wall 685 ft long to assist tows entering and leaving the lock. When the I-wall at the lower lock approach at Lock and Dam 1 was overtopped in the 1984-85 high-water period, shortly after the project became operational, there was major sediment deposition in the vicinity of the lock. Material deposited against the lower miter gates and fell into the lock chamber when the gates were opened. Studies indicated the downstream I-wall should be raised to a higher elevation, and the wall was raised vertically using treated timbers supported by steel H beams. The timber wall extends 900 ft downstream of the miter gates and has successfully reduced the deposition that occurs in the lower approach. Some deposition still occurs, but in smaller amounts, and in areas that can be dredged more easily.

The pool at Lock and Dam 1 is at elevation 40; the dam has 11 tainter gates, and the lock is separated from the dam by an 250-ft nonoverflow section. The upstream and downstream lock approaches at Lock and Dam 1 are separated from the active flow portion of the river up to a specific stage by an earthen embankment and a concrete I-wall, Figures 10.9 and B.2.

The navigation pool at Lock and Dam 2 is at elevation 64; the dam has five tainter gates and a 190-ft uncontrolled crest at elevation 66. This structure was under construction when initial sediment problems occurred at Lock and Dam 1 in 1985, limiting modifications that could be made to Lock and Dam 2 to avoid similar problems. To separate the downstream lock approach from the main river flow, a rock dike was used the same length as the lock wall and at an elevation 10 ft above the lower pool. This configuration was designed to provide a slack-water area for the lower approach and allow some flow near the surface to enter the approach to lessen eddy action. After Lock and Dam 2 went into operation in 1987, navigation conditions in the upper lock approach proved to be difficult, as discussed in Section 9.2.

The navigation pool at Lock and Dam 3 is at elevation 95; the dam has six tainter gates and an uncontrolled weir 315 ft long with crest at elevation 97. The downstream guide wall is on the riverward side of the approach to separate the lower lock approach from the main river channel. Deposition downstream of the miter gate was still a concern, and 3-in drain pipes were installed on 3-ft centers through the lower miter gate sill to provide almost continuous flow to prevent deposition immediately downstream of the gates. This design appeared to be effective and was incorporated in Locks 4 and 5 also (Robertson, 1995).

Locks and Dams 4 and 5 have pool elevations of 120 and 145 ft, respectively. These dams have five tainter gates, a hinged crest gate 100-ft long, and an uncontrolled weir 150-ft long with crest 2 ft above normal pool level. The lower guide wall is on the river side of the approach at both locks.

B.2 Sediment

The Red River drainage basin is approximately 96,000 sq miles, and about 50,000 sq miles is above Denison Dam which traps most sediment from the upper basin. The primary source of the sediment transported on the lower Red River is from bank erosion downstream from Denison Dam (Pinkard, 1995a). The average annual suspended sediment load of the Red River is 32 million tons at Shreveport (mile 228.4) and 37 million tons at Alexandria (mile 88.6). The suspended load is roughly 25 percent fine and very fine sand and 75 percent silt. Bed load is estimated to be less than 10 percent of the total load. Bed material is predominately fine to medium sand, and the material becomes finer in a downstream direction.

Significant sediment deposition problems developed at Lock and Dam 1 in the high-water period following completion of the project in the fall of 1984: in the upstream lock approach; along the riverside lock wall; in the downstream lock approach channel; and in the lock chamber, as shown in Figure B.2.

Deposition in the upstream approach, which was a slack-water area, occurred when flows exceeded 60,000 to 70,000 cfs (the 1-yr frequency flood is 95,000 cfs), appeared to be related to the width needed for safe navigation by tows entering and leaving the upper channel entrance and also by concentration of flow in a deep natural channel along the right bank. A series of four spur dikes was constructed along the upper right bank, Figure B.3, to direct flow toward the left bank. Following construction of the dikes, maintenance dredging in the upper approach decreased significantly, from 1,024,000 cu yds in 1984-85, to 284,000 in 1985-86, and 242,000 in 1986-87. Hydrographs for the three years were comparable (Little, 1987).

Deposition in the lower approach occurred when the tailwater overtopped the downstream I-wall, resulting in eddy action in the lower approach. The I-wall is overtopped for long periods due to backwater from the Mississippi River. Material deposited was primarily very fine sands and silts, with a d_{50} of 0.07 mm. In the downstream approach, there was as much as 20 ft of deposition adjacent to the lower guide wall and 8 to 10 ft around the lower miter gate following the 1984-85 high water, Figure B.2. There was concern that deposition along the riverside lock wall would threaten stability of the wall, and sediment deposition resulted in damage to the lower miter gates. Repairs closed the river to navigation for about three months in 1985.

The elevation of the downstream I-wall (38 ft) was raised by constructing a timber wall, with top elevation of 55 extending 900 ft downstream from the miter gates. After these modifications were made, deposition downstream of the lock decreased substantially, Figure B.4. A profile showing typical deposition in the lower approach in 1985, prior to construction of the timber wall, is compared with deposition in 1987 with the timber wall in place in Figure B.4. While deposition was not completely eliminated in the lower approach by these measures, it was moved downstream to where it is not a threat to the structure and can be easily removed.

Maintenance dredging at Lock and Dam 1 is discussed in Section 10.4.

B.3 Hinged Pool Operation

Hinged pool operation can be used for sediment management as well as to reduce real estate acquisition costs. As flood levels drop, water surface slopes through a pool decrease, and sediment tends to deposit in the middle reach of some pools. Drawing the pool down at the lock and dam increases the water surface slope through the pool, providing better sediment transport. Material tending to deposit in the head end of the pool is transported farther downstream into the pool where depths available for navigation are greater.

Pool hinging to reduce maintenance dredging quantities has been tested in several pools on the Arkansas River (Corps of Engineers, 1987). Results indicated that a hinging operation has the potential to substantially reduce dredging quantities in some pools, but that to maximize benefits it is necessary to determine the optimum time to initiate and terminate dredging for each pool.

Several design factors must be considered where hinged pool operation is planned:

a. The upper gate sill must be set sufficiently low so that navigable depth is provided when the pool is lowered.

b. Velocities and cross currents in the upper lock approach may be more severe than with normal pool operation.

c. Tie-up facilities for tows along the upper approach wall must be usable at the lowered pool level.

d. Port and docking facilities, water intakes, and similar structures just upstream of the dam must be designed to avoid problems resulting from lower pool levels.

e. Rapid pool drawndown may cause bank instability.

f. Operation of the spillway gates is more complex than for normal pool operation, and this could lead to misoperation of the gates.

Locks and Dams 3, 4, and 5 on the Red River are designed for hinged pool operation, but at this time only Lock and Dam 3 is operated as a "hinged pool." A constant pool elevation of 95 ft is maintained during low flows, and as streamflow increases, the water surface at Lock and Dam 3 is drawn down to 89 ft. The water surface at the dam is maintained at this lower level until tailwater begins to control the pool level. Less land is inundated at the head end of the pool with this operation than if the pool were held at normal pool level. Comparative water surface profiles and limits of acquisition of flowage easements with and without hinged-pool operation are shown on Figure B.5.

B.4 Reaeration

Low dissolved oxygen levels below impoundments during summer low-flow periods can be very detrimental to fishery resources, and various measures are employed to alleviate the problem. At Dams 4 and 5 on the Red River, a hinged crest gate is used at one spillway bay to draw warm water from the surface of the pool and discharge it onto a baffled chute, Figure B.6. Turbulence on the chute increases the dissolved oxygen concentration.

B.5 Optimization of spillway design

Lock and Dam 1 was designed to pass the project design flood (100-yr recurrence frequency post-project flood) with one ft of swellhead. A gated dam with 11 spillway bays was needed to meet this criterion, and the widened channel cross section required in the vicinity of the lock and dam to accommodate the structures was a contributing factor to sediment deposition problems immediately after the project went into operation.

For the other four locks and dams upstream, spillway optimization studies were made to compare the cost of each additional tainter gate to costs associated with inundating additional upstream lands with swellheads in excess of one ft. Based on the optimization studies, the four upstream locks and dams were designed with fewer tainter gates than used at Lock and Dam 1, and the dams also included either an uncontrolled or hinged crest gated overflow section, or both (Pinkard, 1995b).

The procedures used in the Red River optimization studies for Lock and Dam 3 are summarized in Attachment B.1. The attachment is a copy of Appendix D to the Corps of Engineers' EM 1110-2-1605, *Hydraulic Design of Navigation Dams*, 1987.

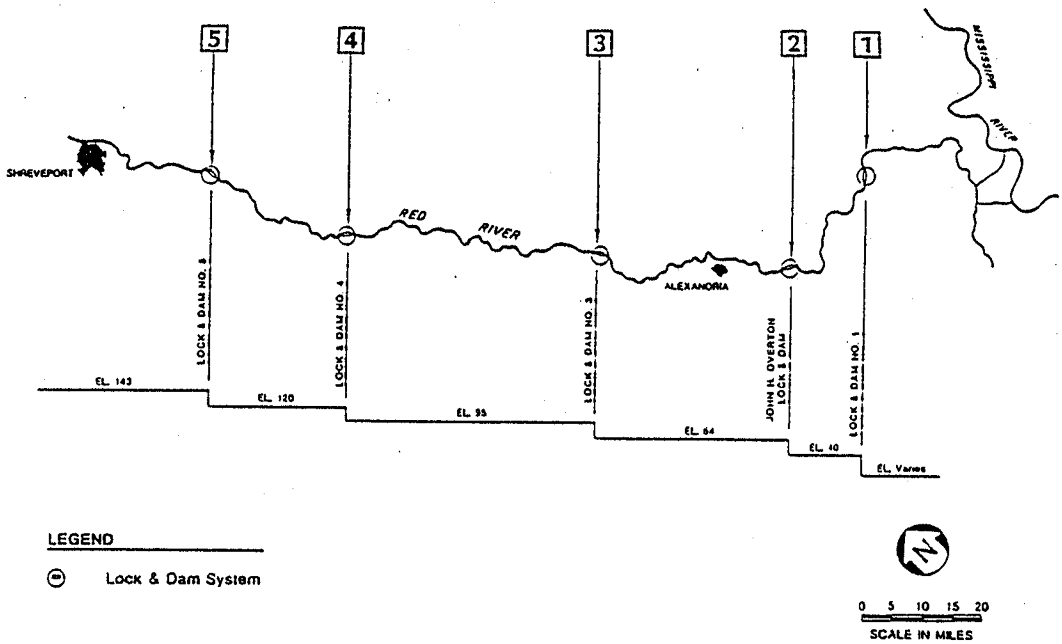


Figure B.1, Red River Waterway, Louisiana, plan and profile (Combs and Espey, 1990).

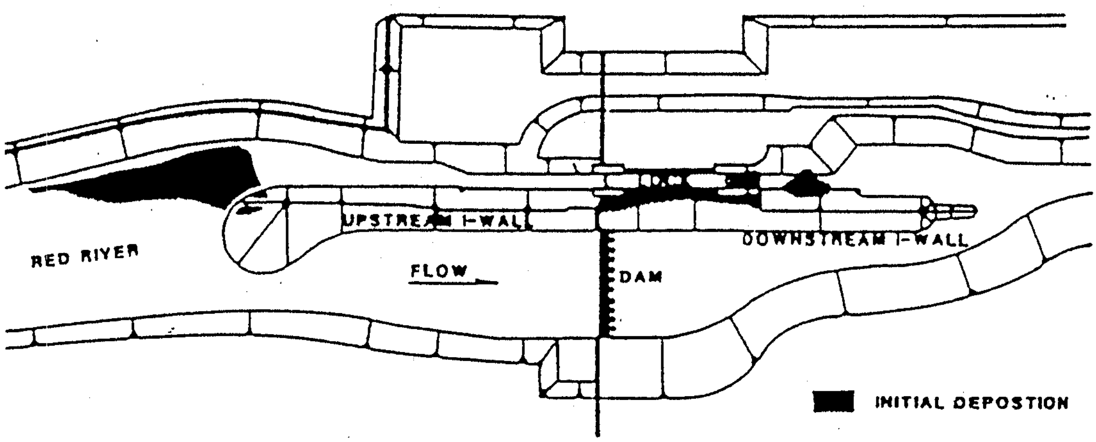


Figure B.2, Initial deposition problem areas Lock and Dam 1, Red River Waterway. (Little, 1987).

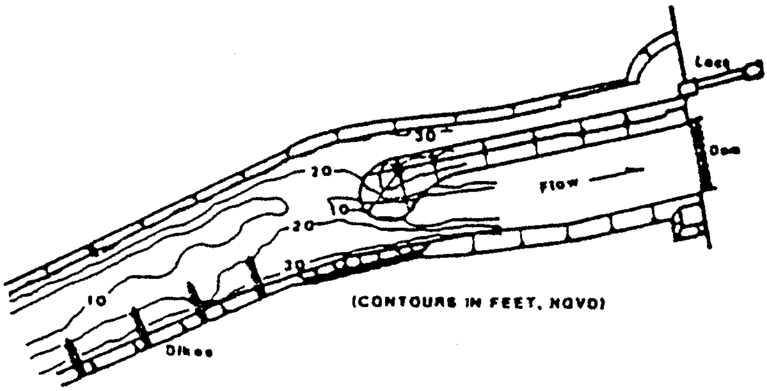


Figure B.3, Bed elevations 20 days after dike construction Lock and Dam 1, Red River Waterway (Little, 1987).

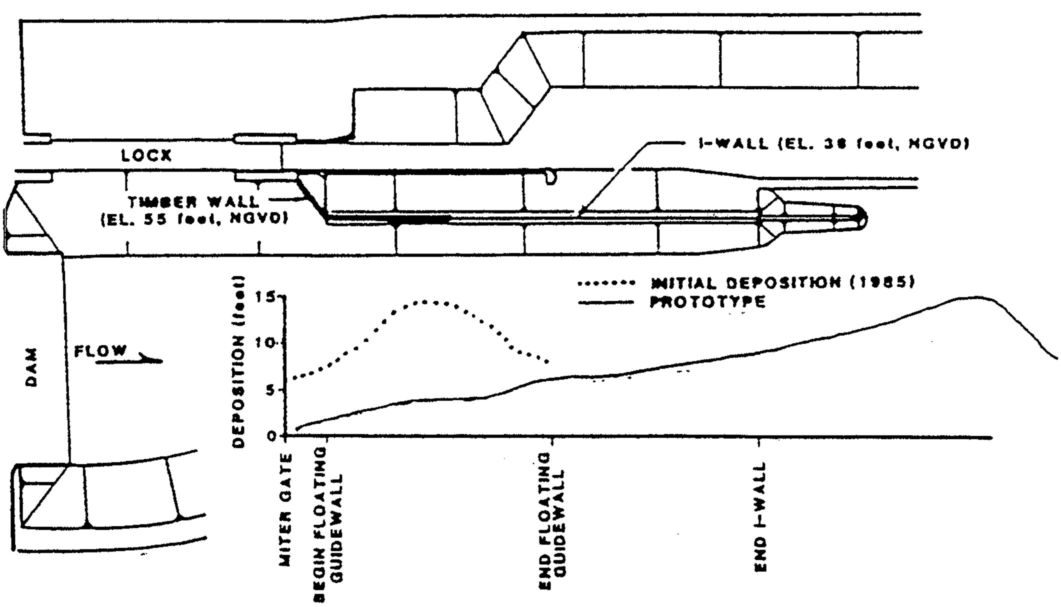
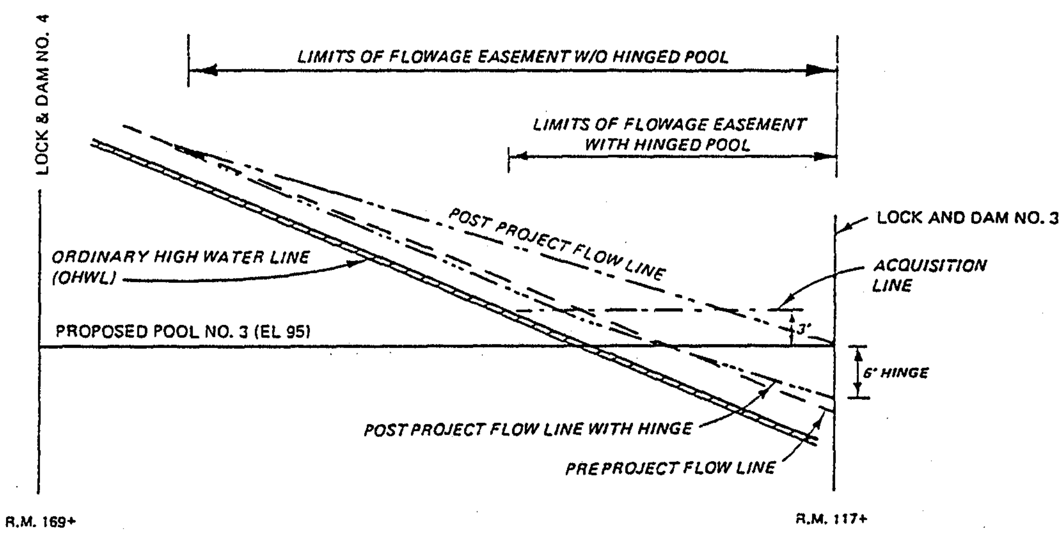
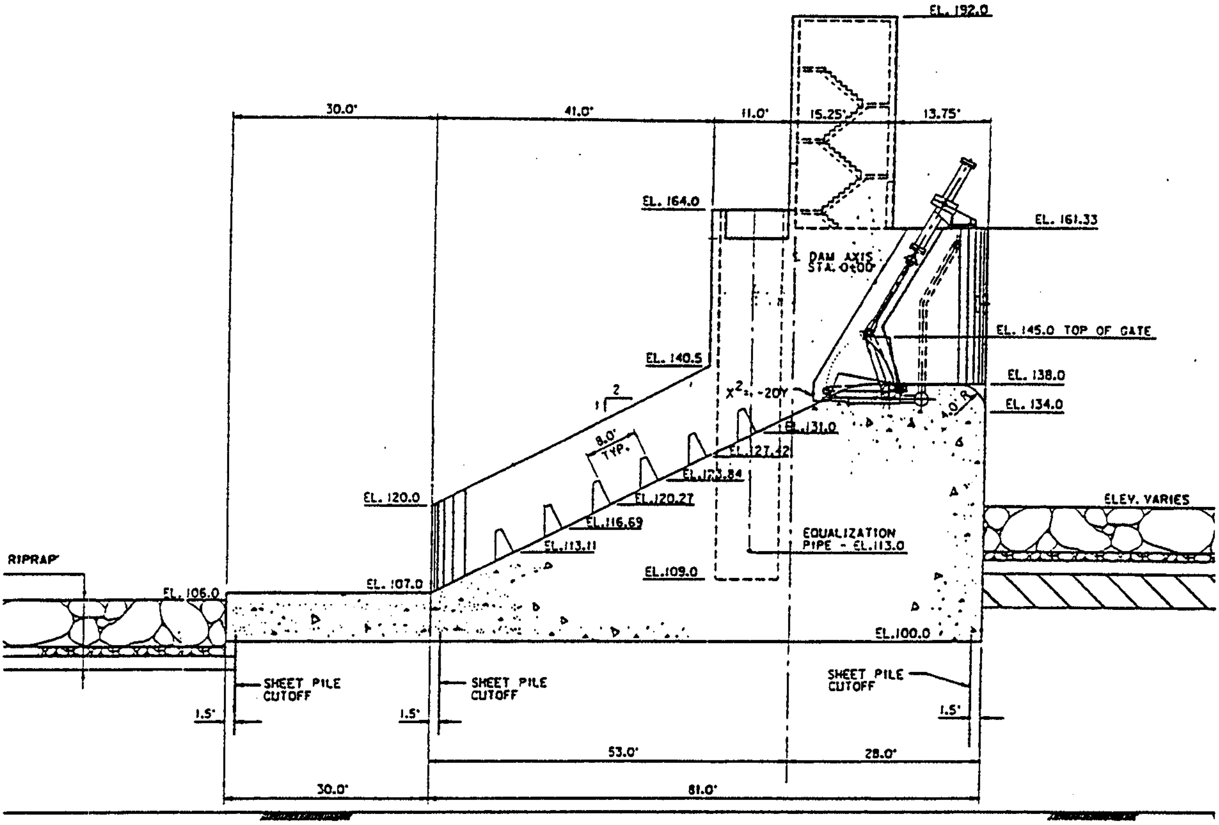


Figure B.4, Deposition below Lock and Dam 2 resulting from hydrograph for period 19 November 1986 to 6 January 1987 (Little, 1987).

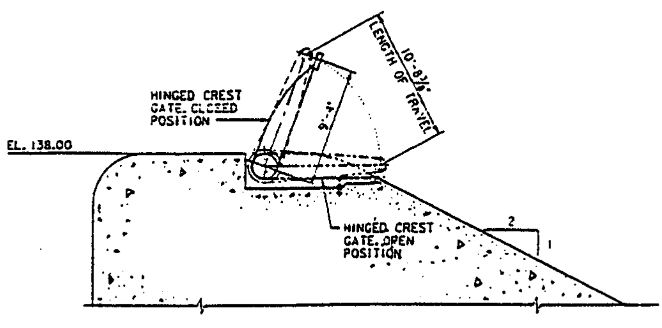


NOTE: Q = 100,000 CFS FOR THIS ILLUSTRATION

**Figure B.5, Hinged pool operation,
Lock and Dam 3, Red River Waterway
(Corps of Engineers, 1987).**



a. Section through baffled spillway chute.



b. Details of hinged gate.

Figure B.6, Hinged gate and baffled spillway chute, Locks and Dams 4 and 5, Red River Waterway (Corps of Engineers).

Attachment B.1. Typical Spillway Optimization Study, Red River, Louisiana (Appendix D, CE EM 1110-2-1605, 12 May, 1987)

1. SCOPE. This appendix summarizes the optimization studies for selection of spillway components. The goal is to select the optimum number of spillway gates and length of overflow dam. The spillway alternatives studied are tabulated in Table D-3.

2. DESIGN GUIDANCE FOR NAVIGATION DAM STRUCTURES.

a. Plans with Gates Only (No Overflow Dam). These plans provide a T-wall dam extending from last gate pier to nonoverflow embankment dam. Length of T-wall dam is governed by excavation slopes for last spillway gate bay and by location of the riverward end of the nonoverflow embankment dam. The landward end of the T-wall dam must be embedded in the riverward end of the nonoverflow embankment dam. The tops of abutments and T-wall dams must be above the headwater for the project design flood plus wave runup. Provide minimum training wall downstream of last gate bay.

b. Overflow Dam Plans with Weir 300-, 600-, and 1,200-foot Crest Lengths. These plans provide concrete overflow dam from the last gate pier to the overflow embankment dam. Length of concrete overflow dam is governed by excavation slopes for last spillway gate bay and by the riverward end of the overflow embankment dam. The overflow embankment dam was extended landward so that total length of concrete overflow plus embankment overflow is 300, 600, 1,200 feet, or other selected lengths. Easy vertical transition from overflow embankment to nonoverflow embankment has been provided. For some instances with four, five, and six gate bays, stone will not resist the overflow velocities on the downstream edge of the embankment crown, and a concrete section must be provided. Minimum training wall downstream of last gate bay must be provided.

c. Spillway Gate Piers. The trunnion anchorage elevation can be the same for all gate arrangements since it is related to tailwater.

d. Riprap. Riprap that is needed for each dam arrangement must be provided. A complete layout plan for each dam arrangement must be developed.

e. Top of Lock Walls. The top of lock walls will be eight feet above the normal upper pool for all gate arrangements. This elevation will provide substantially more than two-foot clearance above the headwater for a 10-year flood for all gate arrangements.

f. Stilling Basins and Gated Weirs. The stilling basin will have the same dimensions in an upstream-downstream direction regardless of the number of gates. The gated crests will also have the same dimensions regardless of the number of bays.

3. FLOWAGE EASEMENTS.

a. Some of the spillways would raise flood heights above preproject elevations. Assume that flowage easements are required on all lands above the ordinary high-water line on which flood heights are increased.

b. The channel realignments on this waterway would reduce the overall river length from the mouth of the Black River (1967 mile 34.2) to Shreveport (1967 mile 278) by 48 miles. This shortening will cause a reduction in flood elevations, and the reduction at the Lock and Dam 3 site is estimated to be 2.2 feet. This postproject reduction of 2.2 feet was taken into account when determining whether a given spillway arrangement would raise postproject flood levels above preproject levels. For example, the six-gate, 315-foot-weir spillway would cause a headwater elevation 2.2 feet above postproject tailwater elevation for the project design flood (PDF). However, this spillway would not raise flood heights since the postproject tailwater elevation is estimated to be 2.2 feet below the preproject tailwater elevation.

c. Table D-2 shows how much various spillway arrangements would raise the PDF (248,600 cfs) above preproject level at the damsite and the land acreages on which the PDF would be raised. The calculations showed that the following spillway arrangements would not raise the PDF above preproject conditions.

Number of Gates	Length of Overflow Dam, feet
4	1,510 and longer
5	935 and longer
6	315 and longer
7	0 and longer
8	0 and longer

d. It is proposed to acquire flowage easements up to elevation 98, which is three feet above the navigation pool elevation and one foot above the top of the overflow dam. When a postproject discharge reaches this headwater elevation at the damsite, the water-surface profile upstream will be higher than the flowage easement elevation 98 throughout Pool 3. The postproject discharge will be 178,000 cfs when the headwater elevation at the damsite is 98, and this discharge has an average recurrence interval of about 33 years.

e. The preproject profile for 178,000 cfs was calculated and compared with the postproject profiles for this discharge for the various spillway arrangements. The postproject profiles for the six-, seven-, and eight-bay spillways were equivalent to or lower than the preproject profile. Since the 178,000-cfs discharge would be only about a foot above the top of the overflow dam, the length of overflow dam does not have a significant effect on the headwater elevation. Table D-1 shows how much various spillway arrangements would raise the 178,000-cfs discharge above preproject level at the damsite and the land acreages on which this discharge would be raised.

4. LEVEE RAISING. The following spillway arrangements would raise the PDF by a foot or more above preproject and would require raising the flood-control levees adjacent to Pool 3 to provide the preproject level of protection.

Number of Bays	Length of Overflow Dam, feet
4	None
4	300
4	600
4	1,200
5	None
5	300
5	600
6	None

The entire length of this levee would be raised by the amount of height that the postproject PDF is raised above preproject at the mouth of Saline Bayou. The levees would be raised to the same height above the postproject PDF as they were above the preproject PDF.

5. COMPARATIVE COSTS. Detailed cost estimates were calculated for each of the alternative spillway arrangements using October 1982 price levels. These estimates are summarized in Table D-3.

6. CONCLUSIONS AND RECOMMENDATIONS.

a. The alternative consisting of a six-bay spillway and 315-foot overflow dam is the least costly considering all costs and is the selected spillway. The lock and dam structure costs for some of the alternatives were less than for the selected plan, but their costs for additional flowage easements and levee raising caused their total costs to be higher.

b. The recommendations for this site-specific study is to proceed with the alternative consisting of six-bay spillway and 315-foot overflow dam design.

TABLE D-1

Spillway Arrangements That Would Raise 178,000 cfs Above Preproject

Spillway Arrangement: No. of Bays	Spillway Arrangement: Length of Overflow Dam, feet	Height of Post-project 178,000 cfs above Pre-project 178,000 cfs at Damsite feet	Flowage Easements Required on Main Stem acres	Flowage Easements Required on Tributaries Approx. acres
4	All	2.0	7,000	6,910
5	All	0.9	7,000	6,910

TABLE D-2

Spillway Arrangements That Would Raise the PDF Above Preproject

Spillway Arrangement		Height of Postproject PDF above Preproject PDF at Damsite, feet	Flowage Easements Required on Main Stem acres	Flowage Easements Required on Tributaries Approx acres
No. of Bays	Length of Overflow Dam, feet			
4	None	5.3	8,500	6,910
4	300	2.8	8,241	6,910
4	600	2.0	8,147	6,910
4	1,200	0.6	7,000	6,910
5	None	2.4	8,273	6,910
5	300	1.2	7,000	6,910
5	600	0.7	7,000	6,910
6	None	1.0	3,328	3,075
6	300	0.2	—	—

TABLE D-3

Comparative Costs

Spillway Alternative		Lock and Dam Structure Costs	Additional Flowage Easement Costs	Levee Raising Cost	Total Comparative Cost
No. of Bays	Length of Overflow Dam, feet	In Dollars Rounded to Nearest Tenth of a Million			
4	0	157.6	11.6	24.7	193.9
4	300	154.8	11.4	12.1	178.3
4	600	156.5	11.3	8.0	175.8
4	1,200	158.1	10.4	Min	168.5
4	1,510*	158.9	10.4	Min	169.3
5	0	163.8	11.4	10.8	186.0
5	300	162.0	10.4	4.9	177.3
5	600	162.4	10.4	Min	172.8
5	935**	163.3	10.4	0	173.7
5	1,200	164.5	10.4	0	174.9
6	0	170.0	4.8	3.4	178.2
6	300	168.0	0	0	168.0
6	315†	168.0	0	0	168.0
6	600	168.6	0	0	168.6
6	1,200	170.7	0	0	170.7
7	0	176.3	0	0	176.3
7	300	174.3	0	0	174.3
7	600	175.9	0	0	175.9
7	1,200	179.3	0	0	179.3
8	0	183.8	0	0	183.8
8	300	182.3	0	0	182.3
8	600	183.8	0	0	183.8
8	1,200	187.6	0	0	187.6

* Structure costs were extrapolated. This alternative would not raise the PDF.

** Structure costs were interpolated. This alternative would not raise the PDF.

† This is the selected alternative. It would not raise the PDF. The six-bay spillway and 315-foot overflow dam was selected over the six-bay spillway and 300-foot overflow dam because the latter alternative would raise flood heights slightly above preproject conditions. No additional costs were shown in the table for additional flowage easements and levee raising for this slight rise in flood heights because they would be of questionable accuracy. However, the 315-foot overflow dam has the advantage of not raising flood heights, while the 300-foot overflow dam could be difficult to defend since it will raise flood heights to some extent.

INDEX

A

Air admission at culvert filling valves, 8-19, 9-8, 9-26
Arkansas River, 8-2 (*see also* Dardanelle Lock and Dam; Robert S. Kerr Lock and Dam; Lock and Dam 17, Verdigris River)
  additional contraction, 10-3
  cofferdams, 8-32
  construction sequence, 10-2, 10-3
  cutoff, typical, 10-11
  dikes, typical system, 10-10
  dredging, 10-2
    initial dredging, 10-3
    maintenance dredging, 10-3, 10-4
      annual volumes, 10-12
      cumulative volume, 10-4
      function of discharge, 10-14
    policy, 10-4
    Pool 2, 10-3, 10-4
  hinged pool operation (*see* Hinged pool operation)
  locks, 8-22, 9-2
    costs, 11-3
    discharge manifolds with baffles, 8-51
  navigation dams, 7-1
  navigation project, 10-10, 10-11
    project plan, 10-11
  pool characteristics, 10-3, 10-4
  sediment load, 10-2
  spillway piers, 7-5
  submerged dikes in upper lock approach, 9-2

B

Barkley Lock, 8-16
Bay Springs Lock, Tennessee-Tombigbee Waterway, 8-7, 8-23, 8-55, 9-6, 9-7, 9-8
  canal surge, 9-7
  construction, 8-34
  downstream canal design, 9-7
  emptying time, 9-7
  filling time, 8-23, 8-24
  floor culverts, 8-23
  hawser stresses, 8-23, 9-7
  intake design, 9-8, 9-25
  layout, 9-21
lock emptying time, 8-23, 9-7
  outlet system, 9-7
  valve opening time, 9-7
  vortex action, 9-8
Bottom longitudinal floor culverts (*see* Longitudinal floor culverts)

C

Canal surges, 9-4
  amplitude
    Bay Springs,Tennessee-Tombigbee Waterway, 9-7
    Verdigris River, 9-8
Canalization, 1-2, 3-1
  Arkansas River, 10-2
  Ohio River, 7-2, 11-1, 11-17
  physical constraint, 2-1
  planning, 2-1
    economic studies, 2-1
    evaluating existing streams, 2-1
    locks, 2-2
    navigation equipment 2-1
    physical constraints, 2-1
    tow size, 2-2
    views of users, 2-2
  Red River Waterway, B-1
  Upper Mississippi River, 10-14, 11-1, 11-18
Channel width (*see* Navigation channel, width)
Choteau Lock and Dam (*see* Locks, Lock and Dam 17, Verdigris River)
Cofferdams, 8-2, 8-29, 8-30, 8-31, 8-32
  Dardanelle Lock and Dam, Arkansas River, 8-2, 8-32

Lock and Dam 26 replacement, Mississippi River, 8-2, 8-31
passing navigation traffic, 8-2
problems during project construction, 8-2
Smithland Locks and Dam, Ohio River, 8-2, 8-30
stages of construction, 8-2
Construction
cofferdams, 8-2
off-channel (in the dry), 8-2, 8-33
Dardanelle Lock and Dam, Arkansas River, 8-2
materials availability, 8-3
Melvin Price Locks and Dam, Mississippi River, 8-2
Red River Locks and Dams, 8-2, 8-33
site access, 8-3
Construction materials, 8-3
Corps of Engineers
Inland Waterways System, 1-1
innovative lock design, 11-3
Cross currents
effect on navigation, 8-1
Culverts (*see* Lock culverts, Lock filling and emptying systems)
Currents
cross currents, 8-1, 9-2, 9-3
hazardous currents, 5-3
lower lock approach, 8-21
ports in lower guard wall, 9-2
ports in upper guard wall, 9-1, 9-2
upper approach, Dardanelle Lock, 9-2
Cutoffs
Arkansas River, 10-11
Red River Waterway, B-1

D

Dams
classifification by head, 3-1
low-head dams, 7-1
wickets, 7-7
Dardanelle Lock and Dam, Arkansas River, 8-2, 8-22, 9-2
baffles, 8-22
cofferdams, 8-32
deposition in lower lock approach, 9-9
filling time, 8-22
hawser stresses, 8-22
intake ports, 8-44
longitudinal floor culverts, 8-53
ports in guard walls, 9-2
sediment, 10-2
submerged dikes in upstream lock approach, 9-2
trashracks, 8-44
wing dike in lower approach 9-9
Depth
minimum depth, 5-1
Dikes
Arkansas River, 10-10
Red River Waterway, B-3, B-7
submerged in upper lock approach, 9-2
Lock and Dam 2, Red River, 9-2
Arkansas River locks, 9-2
Discharge manifolds, 8-49, 8-50, 8-51 (*see also* Lock emptying systems)
Disssolved oxygen levels, 7-2 (*see also* Reaeration)
Double lockage (*see* Lockage time, double lockage)
Dredged material disposal
disposal areas, 10-9
problems, 10-9
Dredging, 10-1
advance maintenance dredging, 10-2
aperiodic dredging, 10-1
Arkansas River (*see* Arkansas River, dredging)
at heads of pools, 4-4
costs, 10-1
environmental impacts, 10-1
initial construction dredging, 10-1
maintenance dredging, 10-1
effect of hinged pool operation, 10-7
effect of reservoir operation, 10-2
Mississippi River, 10-5
Missouri River, 10-6
objectives, 10-1
overdepth dredging, 10-1, 10-2
new construction dredging, 10-1
periodic dredging, 10-1
priorities, 10-1
production rate, 10-1
Red River, 10-6
selecting disposal method and site, 10-1
selecting dredging method, 10-1
Dredging equipment
discharge lines, 10-8
floating lines, 10-8
shore lines, 10-8
hydraulic pipeline (suction) dredges, 10-7, 10-8, 10-17, 10-18
cutterhead, 10-8, 10-18
dustpan, 10-8, 10-18
plain suction, 10-8
sidecasting, 10-8

hopper dredges, 10-8, 10-16
mechanical dredges, 10-7, 10-16
bucket dredges, 10-7
dipper dredges, 10-7
Drift angle of tow, 5-5

E

Eddies in lower lock approach, 9-4
Emergency lock closure (*see* Lock closure facilities)

F

Fish, migratory
mitigation measures, 6-2
Flood releases
tapering off, 10-2
rate of flow reduction, 10-2
Flood stage
effect on dam height, 6-1
hinged-pool operation, 6-1
Flows limiting navigation, 9-1, B-1

G

Gallipolis Locks and Dam, Ohio River
innovative lock design, 11-1
intake manifold, 8-45, 8-46
vortex action, 8-18
Gates (*see* Spillway gates; Lock gates)
Gate sills (*see* Lock gates, gate sills)
Greenup Locks, Ohio River, 8-21, 9-1, 8-40

H

Hazards (*see* Navigation hazards)
Hawser forces, 8-13, 8-19, 8-20
Bay Springs Lock, Tennessee-Tombigbee Waterway, 9-7
Dardanelle Lock, 8-22
disturbances causing, 8-13
limiting criteria, 8-13
model data, 8-13
Head
reverse head, 8-6, 8-8
High-lift locks, 8-22
Bay Springs Lock, Tennessee-Tombigbee Waterway, 8-23
bottom longitudinal systems, 8-22
filling and emptying systems, 8-22
Lower Granite Lock, Snake River, 8-22
Hinged pool operation, 6-4, 10-2
Arkansas River, 1995 floods, 10-7
design factors, B-4
effect on maintenance dredging, 10-7
Red River Waterway, B-3, B-8
Locks and Dams 3, 4, and 5, B-8
Hinged spillway crest, 7-2, B-9
Hydropower at navigation dams, 6-3
flow available for generation, 6-3

I

Ice
accumulation on barges, 8-8
passing ice at navigation projects, 7-5
Upper Mississippi River, 7-5
Illinois River, 7-4
Inland waterways system in U.S., 1-4, 11-7
Innovative lock design (*see* Locks, innovative design)
Intake manifolds (*see* Lock intake manifolds)

K

Kanawha River, 8-5, 11-4, 11-5

L

Lift (*see* Lock lift)
Lockage time
delays, 11-1, 11-4
double lockage, 11-1, 11-6
Lock approach conditions, 5-3, 9-2, 9-3, 9-4
Bay Springs Lock, Tennessee-Tombigbee Waterway, 9-7, 9-23
Dardanelle Lock, Arkansas River, 9-2, 9-10, 9-31
dikes
submerged, 9-2
contraction B-7
eddies in lower approach, 9-4
fill along shore line, 9-3
guide and guard walls, 5-3, 9-31
Lock and Dam 1, Red River Waterway, B-2, B-7

Lock and Dam 2, Red River Waterway, 9-2, 9-11, B-2, B-7
Lock and Dam 17, Verdigris River, 9-8, 9-27, 9-28, 9-29, 9-30
Lock and Dam 52, Ohio River, 9-4, 9-18
Olmsted Lock, Ohio River, 9-4, 9-16, 9-17
Red River Waterway, B-2, B-3, B-7
Robert S. Kerr Lock, Arkansas River, 9-3, 9-12, 9-13, 9-14, 9-15
sediment deposition (*see* Shoaling in lock approaches)
shoaling (*see* Shoaling in lock approaches)
sight distance, 9-1
spur dikes, B-3, B-7

Lock capacity
increasing lock capacity, 11-2, 11-3
Lock and Dam 26, Mississippi River, 11-2
Melvin Price Locks and Dam, Mississippi River, 11-2
Winfield Locks and Dam, Kanawha River, 11-4

Lock chamber size, 8-3

Lock closure facilities
emergency closure, 8-24, 8-25
facilities, 8-25
sills, 8-8
Marmet Lock, Kanawha River, 11-5
Ohio River locks, 8-25
overhead vertical lift gates, 8-25
potential losses, 8-24
sector gates, 8-25
submergible gates, 8-25
stop logs, 8-25
maintenance closure, 8-24
Arkansas River locks, 8-24
low-lift lock facilities, 8-24
submergible gates, 8-24

Lock culverts (*see also* Longitudinal floor culverts)
air entrainment, 8-19, 9-8
air vent size, 9-8
control valves, 8-18, 8-47
tainter gates, 8-18
valve opening times 8-19, 8-47
intakes, 8-18 (*see also* Lock intake manifolds)
surges, 9-8
wall culverts
bulkheads, 8-19
side-port systems, 8-19
unwatering, 8-19

Lock emptying systems, 8-21, 8-49, 8-50, 8-51, 8-52 (*see also* Lock filling and emptying systems)
baffles, 8-22
Bay Springs Lock, Tennessee-Tombigbee Waterway, 9-22, 9-24
common river outlet for two locks, 8-22, 9-4
emptying in lower approach, 8-21, 8-49, 8-50, 9-1
emptying riverward of locks, 8-21, 8-22, 8-49, 8-52, 9-1
Greenup Lock, Ohio River, 8-49, 8-52
lateral diffuser systems, 8-49, 8-50
Greenup Locks, Ohio River, 8-21, 8-49
McArthur Lock, St. Mary's River, 8-21, 8-49
New Cumberland Locks, Ohio River, 8-21, 8-50
Olmsted Locks, Ohio River, 8-52, 9-4
Saint Anthony Falls Lower Lock, Mississippi River, 8-21, 8-50
Snell Lock, St. Lawrence Seaway, 8-21, 8-49

Lock emptying time, 8-39, 9-7

Lock filling and emptying systems, 8-4, 8-5, 8-12, 8-15 (*see also* Lock emptying systems; Locks, innovative lock design)
baffles, 8-51
incremental valve opening, 8-15
longitudinal floor culverts, (*see* Longitudinal floor culverts)
loop culverts, 8-14
low-lift locks, 8-40
modern systems, 8-15 (*see also* Lock filling and emptying systems, Longitudinal floor culverts)
over, between, around lock gates, 8-14
sector gates, 8-14
tainter gates, 8-14
stub culverts, 8-14, 8-40
TVA multiport system, 8-20
valves in gates, short culverts, 8-14
wall culverts and bottom lateral systems, 8-15, 8-40
wall culverts and ports or laterals , 8-15
early conventional systems, 8-15
wall culverts and port systems, 8-15, 8-19, 8-40
port/culvert area ratio, 8-23
port design criteria, 8-19
port elevation, 8-19
port length, 8-19

Lock filling systems (*see* Lock filling and emptying systems)

Lock filling time, 8-41, 8-47
improved tow haulage system, 11-6
innovative design, 11-6
prototype-model relationship, 8-38

Lock floor, 8-6
uplift, 8-6
elevation criteria, 8-6
stabilization of foundation, 8-6
Lock gates, 8-6, 8-34
design load, 8-6
emergency closure, 8-8, 8-25
gate opening time, 5-4
gate sills, 8-6, 8-34
depth/draft ratio, 8-8
effect of ice on clearance, 8-8
sill clearance, 8-7
sill elevation, 8-7, 8-8
Temporary Lock 52, Ohio River, 9-4
lift gates, 8-8
miter gates, 8-6, 8-8, 8-35
operation, 8-6
sector gates, 8-8, 8-14, 8-37
submergible gates, 8-36
tainter gates, 8-8, 8-14
tidal channels,8-8
types, 8-6
Lock intake manifolds, 8-16, 8-17, 8-18
Bay Springs Lock, Tennesse-Tombigbee Waterway, 8-42
bulkheads, 8-17
Dardanelle Lock, Arkansas River, 8-44
Gallipolis Locks, Ohio River, 8-45, 8-46
head loss coefficients, 8-43
Ice Harbor Lock, Snake River, 8-42
innovative design, 11-3
passing drift and ice, 8-16
port/culvert area ratio, 8-16
port size, 8-16
port throat area, 8-16
Saint Anthony Falls Lower Lock, Mississippi River, 8-42
Siamese intakes, 8-16, 8-40
submergence, 8-17
trash racks, 8-17, 8-44
unwatering, 8-17
upper gate sill intakes, 8-16, 11-3, 11-5
vortex action, 8-17
Lock lift, 3-1, 8-4
high-lift locks, 3-1, 8-22
intermediate lift locks, 3-1, 8-4
low lift locks, 3-1, 8-4
Lock walls, 8-9, 8-34
approach walls, 8-11, 8-12
ports to reduce cross currents, 9-2
types, 8-12
culvert discharge walls, 8-11
culvert intake walls, 8-10
gate bay walls, 8-10, 8-11
guard walls, 5-3, 8-11, 9-2
ports, 9-2
guide walls, 5-3, 8-11, 9-3
flow distribution through ports, 9-3
Lock and Dam 1, Red River Waterway, B-2, B-6
Lock and Dam 2, Red River Waterway, 9-2, 9-3, B-2, B-6
Lock and Dam 3, Red River Waterway, B-2
ports, 9-3
Robert S. Kerr Lock, Arkansas River, 9-3
height, 8-5, 8-9, 8-10, B-1
Red River Waterway, B-1
lock chamber walls, 8-10
sight distance, 8-1
types, 8-4, 8-12
criteria, 8-4
walls categorized by purpose, 8-9
wall port systems, 8-16, 8-19 (*see also* Wall ports)
port design criteria 8-19
port elevation 8-19
port length, 8-19
Locks
approach conditions (*see* Lock approach conditions)
access during floods, 8-10
Bay Springs Lock, Tennessee-Tombigbee Waterway, 8-23, 8-55
Bankhead Lock, Black Warrior River, Alabama, 8-54
classification by lift, 3-1
closure during floods, 8-9
combination type, 8-5
costs, 11-3, 11-6
cushion of water, 8-5, 8-19
Dardanelle Lock and Dam, Arkansas River, 8-22
depth, 8-5, 8-6
sill elevation, 8-6
tow squat, 8-7
design criteria, 8-3
lock size, 8-3
number of locks, 8-3
dimensions in United States, 8-3
double lockage 11-1, 11-6
Winfield Locks and Dam, Kanawha River, 11-4
dry-dock type, 8-4, 8-5

filling criteria, 8-19
filling time, 8-5, 8-13
    optimum filling time, 8-8
Greenup Locks, Ohio River, 8-21, 9-1
hazardous approach conditions, 9-1
innovative lock design
    Corps of Engineers test program, 11-3
    culverts on lock floor, 11-3
    faster valve operating speed, 11-6
    guide and guard walls, 11-4
    improved tow haulage system, 11-6
    Marmet Lock and Dam, Kanawha River, 11-4, 11-5
    McAlpine Lock and Dam, Ohio River, 11-4, 11-5
    Monongahela No. 4 Lock and Dam, 11-4, 11-5
    Mississippi River locks, 11-4, 11-5
    need for, 11-3
    thin lock chamber walls, 11-3
    Winfield Locks and Dam, Kanawha River, 11-4, 11-5
intake manifolds (*see* Lock intake manifolds)
floor (*see* Lock floor)
gates (*see* Lock gates)
gravity locks, 8-4
guard walls, 5-3 (*see also* Lock approach walls)
guide walls, 5-3 (*see also* Lock approach walls)
Kanawha River, 8-5, 11-4, 11-5
lift, 5-3, 8-4 (*see also* Lock lift; High-lift locks; Medium-lift locks; Low-lift locks)
location, 8-1
    criteria, 8-1, 9-1
    channel width, 9-1
    cross currents, 8-1, 9-1
    sediment transport and deposition, 9-9
    sight distance, 8-1, 9-1
lockage time, 3-2, 11-6
Lock and Dam 17, Verdigris River, 9-8
Lock and Dam 26, Mississippi River, 11-2
Lock and Dam 26 replacement, Mississippi River, 8-28
    cofferdams, 8-31
Lower Granite Lock and Dam, Snake River, 8-22, 8-41
Olmsted Locks and Dam, Ohio River, 7-2, 8-22, 9-1, 9-4
Melvin Price Locks, Mississippi River, 8-1
mooring facilities, 8-12
multiple locks, 8-3
number of locks, 8-3
operation, 3-2, 3-3
Panama Canal Locks, 8-11
Red River Waterway, B-1
    Lock and Dam 1, B-2, B-3, B-6
    Lock and Dam 2, B-2, B-6
    Lock and Dam 3, B-2, B-4
    Lock and Dam 4, B-4
    Lock and Dam 5, B-4
reinforced concrete locks, 8-4
reverse head, 8-6, 8-8, 8-14
Saint Anthony Falls Locks, Mississippi River, 8-8
    Saint Anthony Falls Lower Lock
        lock miter gates, 8-35
        submersible tainter gate, 8-36
sills (*see* Lock gates, gate sills)
size, 3-2
Snell Lock, Saint Lawrence Seaway, 8-40
steel sheet piling locks, 8-5
temporary locks, 8-5
Temporary Lock 52, Ohio River, 9-4
"twin locks", 8-1
types, 8-4
United States locks, 3-1, 3-2
walls (*see* Lock walls)
Locks and dams, 3-1
    site criteria, 3-1, 4-1, 4-2, 4-3
    in bends, 8-29
    layout for low-lift dams, 8-26, 8-27
Locks and Dams 50 and 51, Ohio River, 8-2
Longitudinal floor culverts, 8-15, 8-16, 8-53, 8-54, 8-55
    baffles along walls and between culverts, 8-22, 8-23
    Bankhead Lock, Black Warrior River, 8-54
    Bay Springs Lock, Tennessee-Tombigbee Waterway, 8-55
    combining culverts, 8-23
    crossover culverts, 8-22, 8-23
    culvert area ratio, 8-23
    culvert distribution (location), 8-23
    Dardanelle Lock, Arkansas River, 8-22, 8-53
    filling and emptying times, 8-24
    innovative design, 11-3
    Lower Granite Lock, Snake River, 8-41
    model data, 8-24
    "over and under" system, 8-23, 8-54, 8-55
    port/distribution culvert area ratio, 8-23
    port manifold location, 8-23
    ports, 8-23
        access for inspection and maintenance, 8-23
        size, 8-23

"side by side" system, 8-22, 8-53
Low-head dams
flow regimes, 7-3, 7-10
lock and dam layout, 8-27
overflow embankments, 7-2
spillways, 7-2
swellhead, 7-2
typical cross section, 4-4
Low-lift locks, 3-1
filling and emptying systems, 8-15, 8-16
layout, 8-27
Lower Granite Lock, Snake River, 8-22
floor culverts, 8-22

M

Maintenance dredging (*see* Dredging, maintenance dredging)
Maintenance of navigation structures, 11-2
Lock and Dam 26, Mississippi River, 11-2
Marmet Lock and Dam, Kanawha River, 11-4
Marseilles Lock and Dam, Illinois River, 7-4
Mc Alpine Lock and Dam, Ohio River, 11-4
Medium lift locks, 8-1
filling and emptying systems, 8-15, 8-16
Melvin Price Locks and Dam, Mississippi River (replacement for Locks and Dam 26), 8-1, 8-28, 11-2, 11-3
costs, 11-3
Mississippi River, 8-1, 8-2, 8-15, 11-1
cofferdams, 8-31
dredging, 10-5
flood of 1993, 8-9
ice, 7-5, 8-8
Locks and Dam 26 replacement, Mississippi River (Melvin Price Locks and Dam), 8-1, 8-28
Melvin Price Locks and Dam, 8-1, 8-28
Saint Anthony Falls locks
discharge laterals, 8-21
Upper lock, 8-35
tainter gates, 8-8
Upper Mississippi River, 7-5
canalization, 10-14, 11-1
traffic, 11-2, 11-8
Mitigation measures
migratory fish, 6-2
Models
Bay Springs Lock, Tennessee-Tombigbee Waterway
air entrainment, 9-8
canal surge, 9-7
filling and emptying time, 9-7
intake design, 9-8
outlet diffuser design, 9-8
valve operating time, 9-7
vortex action, 9-8
Dardanelle Lock and Dam, 9-2
effect of wing dikes on shoaling in lower approach, 9-9
innovaative design models, 11-3, 11-4
intake models, 11-5
Lock and Dam 2, Red River, 9-2
longitudinal floor culverts
filling time, 8-24
emptying time, 9-7
navigation conditions in lock approaches, 9-2, 9-3
Olmsted Locks outlet works, 9-4
riprap stability at outlet, 9-4
Robert S. Kerr Lock and Dam, Arkansas River, 9-2
spillway models, 7-3
surge conditions, 9-7
tow squat, 9-6
Monongahela River No. 4 Lock and Dam, 11-4, 11-5
Mosquito control, 6-2, 6-5

N

Navigation channel
authorized dimensions, 10-10
overdepth dredging 10-10
width, 5-2, 5-5
width in bends, 5-5
Navigation dams, 7-1 (*see also* Low-head dams)
Arkansas River, 7-1, 7-8
effect of flood heights, 6-1
fixed, 7-1
height, 6-1
movable, 7-1, 7-6, 7-8
Babout self-tripping wicket, 7-6
bear-trap dam, 7-6
Chanoine wicket, 7-6
Chanoine-Pescaud wicket, 7-6
Olmsted Locks and Dam, Ohio River, 7-7
wicket gates, 7-7
navigable pass, 7-1
Red River, 7-1
site selection affected by location of water intakes, sewer outfalls, etc., 6-1, 6-2

spacing, 3-3
typical low-head dam, 4-4
wicket gates, 7-1, 7-7
Navigation development
canalized, 1-2
canals, 1-2
cost sharing, 11-3
Federal funding, 11-3
in United States, 11-1
Mississippi River, 11-1
Ohio River, 11-1
open-river, 1-2
Navigation equipment
design tow size, United States, 3-2
tow size (*see* Tow size)
terminals, 5-4
Navigation locks (*see* Locks)
Navigation hazards, 9-1
currents, 9-1
Dardanelle Lock and Dam, Arkansas River, 9-2
expansion eddies, 9-1
flood stages, 9-1
high velocities, 9-1
lock emptying system, 9-1
lower approach, 9-1
Lock and Dam 2, Red River, 9-2
power plant discharges, 9-1
sediment, 9-1
shoaling, 9-1
spillway discharges, 9-1
surges, 9-1
tow squat, 9-1
Navigation pass, 7-1
Navigation pool level
effects on fish and wildlife, 6-2
effects on water supply intakes, 6-1
effects on local drainaage, 6-1
operation for vector control, 6-2, 6-5
Navigation projects
components of projects, 1-3
channel width, 5-2
criteria for design, 5-1, 5-2, 5-3, 5-4
design requirements, 6-1
hazardous currents, 5-3
hydropower development, 6-3
minimum depth, 5-21
objectives in developing, 1-1
recreation use of navigation pools, 6-3
transit time, 5-3
terminal facilities, 5-4
ways to develop, 1-1
Navigation structures
site selection, 4-1, 4-2, 4-3
New Cumberland Locks, Ohio River, 8-21

O

Ohio River
canalization, 7-2, 11-1
Gallipolis Locks vortex action, 8-18
Greenup Locks, 8-40
Olmsted Locks and Dam, 7-2, 8-22, 9-1, 9-4
Smithland Locks, 8-2, 8-30
Temporary Lock 52, 9-4
traffic, 11-1, 11-8
Olmsted Locks and Dam, Ohio River, 7-2, 7-9, 8-22, 9-1, 9-4
emptying system, 9-4
layout, 7-9
Open-river spillway operation, 10-3
Overflow embankments, 7-2

P

Panama Canal Locks, 8-11
Planning
canalization, 2-1
project life, 2-1
site selection, 4-1, 4-2, 4-3
Ports
deflectors, 8-48
for 110-ft lock, 8-48
in approach walls to reduce currents, 9-2
in upper guard wall, Dardanelle Lock, 9-2
in upper guide wall, Lock and Dam 2, Red River, 9-2
TVA multiport system, 8-48

R

Reaeration, 7-2, B-4
Red River Waterway, 7-1, 7-3, 7-4, 9-3, B-1
costs, 11-3
hinged crest spillway, 7-2
hinged pool operation (*see* Hinged pool operation)
Lock and Dam 1
layout, 10-15
sediment deposition (*see* Sediment deposition, Red River Waterway)
submersible pump, 10-6
Lock and Dam 2

ports in upper guide wall, 9-2
submerged dikes in upstream approach, 9-2
locks, B-1, B-2
project description B-1, B-6
reaeration of spillway discharge, 7-2, B-4
sediment deposition (*see* Sediment deposition, Red River Waterway)
sediment load, B-2
spillways, B-1, B-4, B-5, B-9
design optimization, B-4, B-5, B-10
stilling basins B-1
Reoxygenation (*see* Reaeration)
Robert S. Kerr Lock and Dam, Arkansas River, 9-3
cross currents, 9-3
eddies in lower lock approach, 9-4
ports in upper guide wall, 9-3
Reservoir operating criteria
to reduce shoaling, 10-2

S

Saint Anthony Falls Locks, Mississippi River, 8-8
Lower Lock discharge laterals, 8-21
tainter gates, 8-8
Upper lock, 8-35
Vortex action at Lower Lock, 8-17
Saint Lawrence Seaway, 8-40
Snell Lock emptying system, 8-21
Sector gates (*see* Lock gates, sector gates)
Sediment deposition (*see also* Shoaling in lock approaches)
Red River Waterway
at miter gates, 10-6, B-2, B-6
drains in sill to prevent, B-2
in lock chamber, 10-6
Lock and Dam 1, B-2, B-3, B-6
1984. 1-6
1990, 10-6
Lock and Dam 2, B-7
Sheet piling locks, 8-5
Shoaling in lock approaches, 9-9
Dardanelle Lock lower approach, 9-9
effect of ports in upsteam guide wall, 9-9
effect of wing dikes, 9-9
in lower lock approach, 9-9
in upper lock approach, 9-9
minimized by reservoir operation, 10-2
minimized by hinged pool operation, 10-7
Red River Waterway, B-3
Lock and Dam 1, B-3, B-6
Sight distance (*see* Locks, location)
Sills (*see* Lock gates, sills)
Site access for construction and operation, 8-3
Smithland Locks and Dam, Ohio River, 8-2, 8-30
Snake River, 8-23 (*see also* Lower Granite Lock)
Snell Lock, Saint Lawrence Seaway
emptying system, 8-21
Spillways
baffled spillway, 7-3, B-4, B-9
cross currents at locks, 8-1
design optimization, B-4, B-5, B-10
flow regimes, 7-3
hinged crest, 7-2, B-9
hydraulic models, 7-3
low-lift navigation dams, 7-2
passing ice and debris, 7-3, 7-4, 7-5
reaeration, 7-2, B-4
Red River Waterway dams, B-1
uncontrolled crests, B-1
Spillway gates
flap gates, 7-14
hinged crest gates, 7-2, 7-4, 7-14, B-4, B-9
Red River Waterway locks and dams, 7-2, B-4
roller gates, 7-4, 7-13
submergible gates, 7-3
Illinois River, 7-4
Marseilles Lock, Illinois River, 7-12, 7-13
tainter gates, 7-3, 7-11
types, 7-3
vertical lift gates, 7-4, 7-14
Spillway piers
ogival pier nose, 7-8
pier shape, 7-5, 7-8
semicircular pier nose, 7-8
Squat (*see* Tow squat)
Streamflow
maximum navigable flow, 8-9
Submerged dikes (*see* Dikes)
Submersible pump to remove sediment, 10-6, 10-7

T

Tainter gates
spillway gates, 7-3, 7-8
"reverse" tainter gates in culverts, 8-18
Tennessee-Tombigbee Waterway, 8-7, 9-6 (*see also* Bay Springs Lock)
canal surge, 9-7
Tennessee Valley Authority
filling and emptying systems, 8-15
multiport system, 8-20, 8-48
Terminal facilities
site criteria, 5-4

Towboat power, 9-5, 9-8
    Kort nozzles, 9-5
    Ohio River, 9-5
    Verdigris River, 9-8
Tow size
    Arkansas River, 2-2
    Kanawha River, 11-4
    Lower Mississippi River, 2-2, 2-3
    Missouri River, 2-2
    Ohio River, 11-1
    Red River Waterway, B-1
    United States, 3-2
    Upper Mississippi River, 2-2, 2-3, 11-1
    Verdigris River, 9-8
Tow squat
    effect of lock entering speed, 9-6
    effect of open emptying valves, 9-6, 9-19, 9-20
    effect of propeller speed, 9-6
    types of squat
        displacement squat, 9-5
        moment squat, 9-6, 9-19
        piston squat, 9-5, 9-19
        propeller squat, 9-6
    Verdigris River
        effect of channel deepening, 9-9, 9-29, 9-30
        effect of channel widening, 9-8, 9-30
Trashracks, 8-17

V

Valve opening time, 8-47, 9-7, 11-6
    Bay Springs Lock, Tennessee-Tombigbee Waterway, 9-7
Vector control, 6-2, 6-5
Velocities
    in lower lock approach, 9-3
    limiting navigation B-1
Verdigris River, 9-8, 9-27
Vortex action
    Bay Springs Lock, 9-8
    Gallipolis Locks, Ohio River, 8-18
    lock intake ports, 8-17
    model studies, 8-17, 8-18
    port submergence, 8-17
    Saint Anthony Falls Lower Lock, 8-17

W

Walls (*see* Lock walls)
Wall culverts (*see* Lock filling and emptying systems)
Wall ports
    design criteria 8-19
    lock filling time, 8-16
    port elevation, 8-19
    port length, 8-19
Waterborne commerce
    Ohio River, 11-1, 11-8
    Upper Mississippi River, 11-2, 11-8
Waterways Experiment Station
    innovative lock design studies, 11-3, 11-4
Wicket gates (see Dams, wickets)