ASCE Manuals and Reports on Engineering Practice No. 94

Inland Navigation: Locks, Dams, and Channels

ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE Manuals and Reports on Engineering Practice No. 94

Inland Navigation: Locks, Dams, and Channels

Bruce L. McCartney, Chair
John George
B.K. Lee
Mark Lindgren
Frank Neilson

Published by
ASCE American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, Virginia 20191-4400

Abstract::
Inland Navigation: Locks, Dams, and Channels (ASCE Manuals and Reports on Engineering Practice No. 94) was prepared by a task committee of the Waterways Committee, which is part of the Waterways, Ports, Coastal, and Ocean Division. The Manual provides information on planning, design, construction, and operation of the US waterways used by barge traffic. Most of the information comes from design criteria and more than 100 years of experience of the US Army Corps of Engineers. The Corps has built more than 220 lock and dam projects on US waterways and maintains more than 25,000 miles of inland navigation channels. This Manual also includes an inventory of the Corps Lock and Dam projects and six case histories.

Library of Congress Cataloging-in-Publication Data

Inland navigation : locks, dams, and channels / Bruce L. McCartney ... [et al.].
p. cm.—(ASCE manuals and reports on engineering practice ; no. 94)
Includes bibliographical references (p.).
ISBN 0-7844-0320-1
1. Inland navigation—United States. I. McCartney, Bruce L. II . Series.
TC623.I55 1998
627'.1'0973—dc21 98-13187
CIP

Library of Congress Catalog Card No: 98-13187
ISBN 0-7844-0320-1
Manufactured in the United States of America

MANUALS AND REPORTS ON ENGINEERING PRACTICE

(As developed by the ASCE Technical Procedures Committee, July 1930, and revised March 1935, February 1962, and April 1982)

A manual or report in this series consists of an orderly presentation of facts on a particular subject, supplemented by an analysis of limitations and applications of these facts. It contains information useful to the average engineer in his everyday work, rather than the findings that may be useful only occasionally or rarely. It is not in any sense a "standard," however; nor is it so elementary or so conclusive as to provide a "rule of thumb" for non-engineers.

Furthermore, material in this series, in distinction from a paper (which expressed only one person's observations or opinions), is the work of a committee or group selected to assemble and express informaton on a specific topic. As often as practicable the committee is under the direction of one or more of the Technical Divisions and Councils, and the product evolved has been subjected to review by the Executive Committee of the Division or Council. As a step in the process of this review, proposed manuscripts are often brought before the members of the Technical Divisions and Councils for comment, which may serve as the basis for improvement. When published, each work shows the names of the committees by which it was compiled and indicates clearly the several processes through which it has passed in review, in order that its merit may be definitely understood.

In February 1962 (and revised in April 1982) the Board of Direction voted to establish:

> A series entitled "Manuals and Reports on Engineering Practice," to include the Manuals published and authorized to date, future Manuals of Professional Practice, and Reports on Engineering Practice. All such Manual or Report material of the Society would have been refereed in a manner approved by the Board Committee on Publications and would be bound, with applicable discussion, in books similar to past Manuals. Numbering would be consecutive and would be a continuation of present Manual numbers. In some cases of reports of joint committees, bypassing of Journal publications may be authorized.

MANUALS AND REPORTS OF ENGINEERING PRACTICE

No.	Title
13	Filtering Materials for Sewage Treatment Plants
14	Accommodation of Utility Plant Within the Rights-of-Way of Urban Streets and Highways
34	Definitions of Surveying and Associated Terms
35	A List of Translations of Foreign Literature on Hydraulics
37	Design and Construction of Sanitary and Storm Sewers
40	Ground Water Management
41	Plastic Design in Steel: A Guide and Commentary
45	Consulting Engineering: A Guide for the Engagement of Engineering Services
46	Report on Pipeline Location
47	Selected Abstracts on Structural Applications of Plastics
49	Urban Planning Guide
50	Planning and Design Guidelines for Small Craft Harbors
51	Survey of Current Structural Research
52	Guide for the Design of Steel Transmission Towers
53	Criteria for Maintenance of Multilane Highways
54	Sedimentation Engineering
55	Guide to Employment Conditions for Civil Engineers
57	Management, Operation and Maintenance of Irrigation and Drainage Systems
59	Computer Pricing Practices
60	Gravity Sanitary Sewer Design and Construction
62	Existing Sewer Evaluation and Rehabilitation
63	Structural Plastics Design Manual
64	Manual on Engineering Surveying
65	Construction Cost Control
66	Structural Plastics Selection Manual
67	Wind Tunnel Model Studies of Buildings and Structures
68	Aeration: A Wastewater Treatment Process
69	Sulfide in Wastewater Collection and Treatment Systems
70	Evapotranspiration and Irrigation Water Requirements
71	Agricultural Salinity Assessment and Management
72	Design of Steel Transmission Pole Structures
73	Quality in the Constructed Project: A Guide for Owners, Designers, and Constructors
74	Guidelines for Electrical Transmission Line Structural Loading
75	Right-of-Way Surveying
76	Design of Municipal Wastewater Treatment Plants
77	Design and Construction of Urban Stormwater Management Systems
78	Structural Fire Protection
79	Steel Penstocks
80	Ship Channel Design
81	Guidelines for Cloud Seeding to Augment Precipitation
82	Odor Control in Wastewater Treatment Plants
83	Environmental Site Investigation
84	Mechanical Connections in Wood Structures
85	Quality of Ground Water
86	Operation and Maintenance of Ground Water Facilities
87	Urban Runoff Quality Manual
88	Management of Water Treatment Plant Residuals
89	Pipeline Crossings
90	Guide to Structural Optimization
91	Design of Guyed Electrical Transmission Structures
92	Manhole Inspection and Rehabilitation
93	Crane Safety on Construction Sites
94	Inland Navigation: Locks, Dams, and Channels

TABLE OF CONTENTS

This page intentionally left blank

Chapter 1

INTRODUCTION

1.1 PURPOSE

This Manual provides guidance in the planning, design, construction, and operation of shallow-draft waterways, factors that should be considered, and solutions that have been successful in avoiding or eliminating undesirable conditions. This Manual is intended to provide an overview of the design process and operation of waterways used by barge traffic. The numerous references in the Bibliography provide more detail and examples that can be used to supplement this Manual. This Manual consolidates information from many sources with much of the information coming from recent manuals published by the US Army Corps of Engineers.

1.2 HISTORICAL DEVELOPMENT

Inland waterways have always provided a link for agricultural products and manufactured goods from the interior to the coastal ports of the world. The evolution of inland waterways in the United States started with canoes and rafts then continued with riverboats either rowed or pulled in natural rivers. The next major phase was steam-powered vessels on natural rivers. This mode of navigation was limited by excessive currents during floods, limited depths during low flow seasons, and shifting channels.

These limitations on navigation prompted the development of controlled waterways with locks and dams. The dams created reservoirs with predictable channel depths and the locks provided a means to allow safe passage from one pool to the next. Early locks had small chambers and low lifts. Filling and emptying were normally end-filling methods. These locks were filled by a slight opening of the upstream gate or opening small valves in the upstream gate. An example of this type of filling system is the Willamette

Falls Lock shown in Figure 1-1. The lock consists of 4 chambers. Each chamber is 40 ft wide and 210 ft long. Each lift is about 10 ft for a total lift of 41 ft. The maximum vessel draft is 6 ft. These locks were built in 1872 and are still operated by the US Army Corps of Engineers for both commercial and recreation traffic. More details and photos of this lock are provided in Chapter 20.

These early locks were replaced with large locks with higher lifts. An example of the newer locks is the John Day Lock on the Columbia River. This lock has a lift of 110 ft and the chamber is 86 ft wide and 669 ft long. Figure 1-2 shows the John Day Lock. At the present time the inland waterway system in the United States consists of 25,000 miles of inland waterways, over 220 navigation lock and dam sites, and lock lifts in excess of 100 ft. The largest locks in the US system are 1200 by 110 ft for barge traffic and 1350 by 80 ft for ship traffic. The US inland waterway system is shown in Figure 1-3.

1.3 BACKGROUND

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

1.4 SCOPE

This Manual provides an overview of the essential elements of an inland waterway navigation system. Due to the experience of the Task Committee, the main emphasis of this Manual is the design of navigation, locks, dams, and channels. Other subjects presented in less detail are environmental design, economic analysis, construction, operation, and maintenance. It is the hope of this Task Committee that these related subjects with less coverage could be developed into complementary manuals to provide an in-

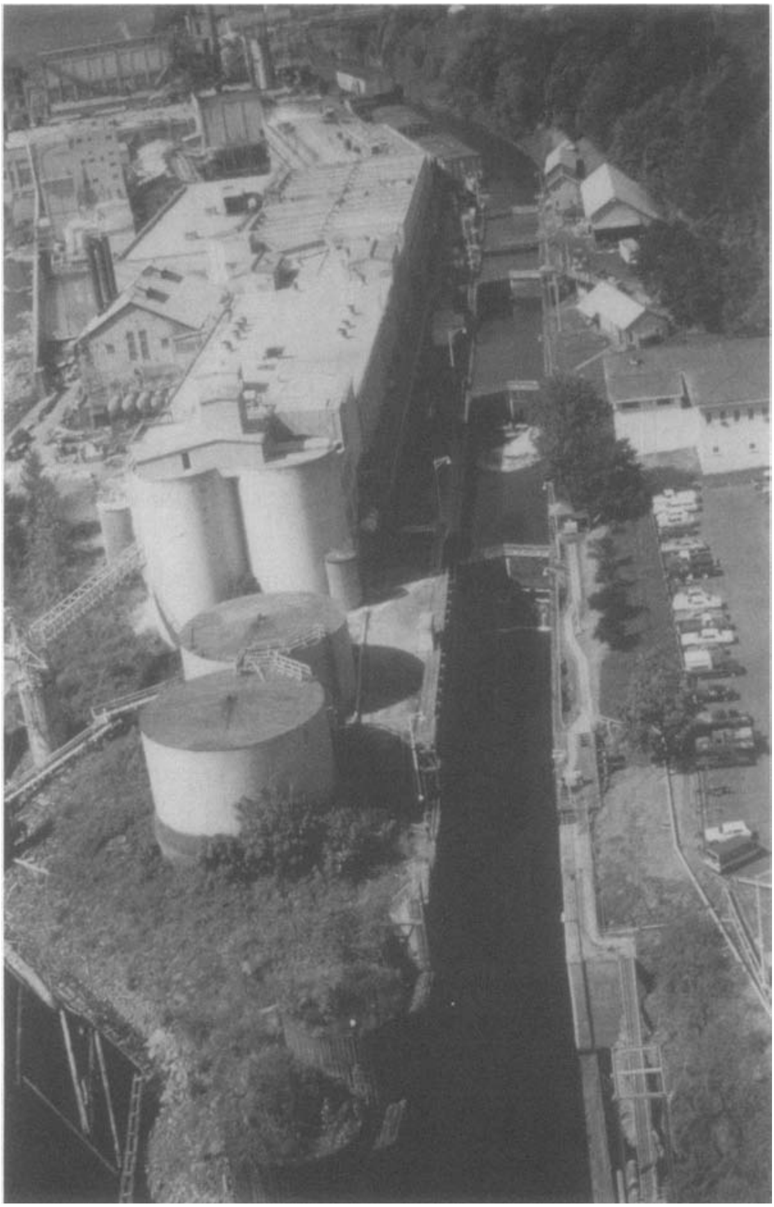


FIGURE 1-1. Willamette Falls Locks, Oregon City, OR.

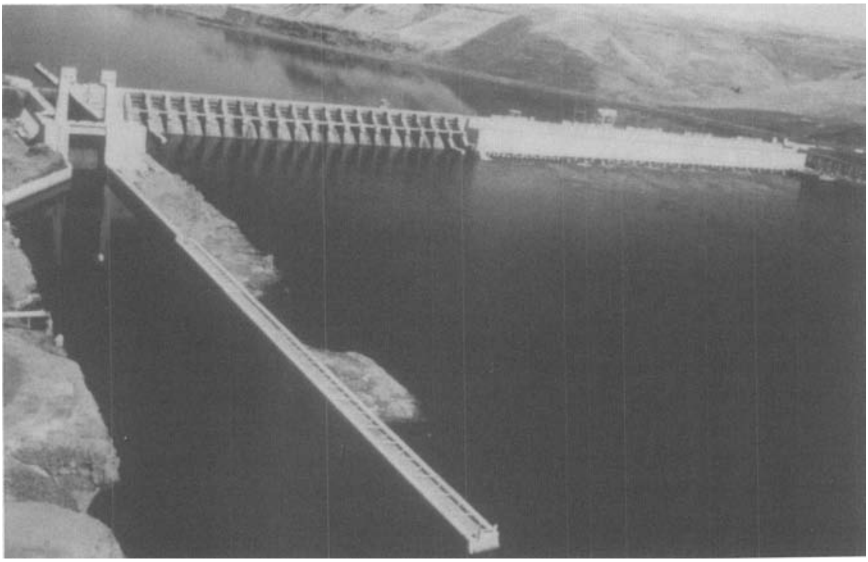


FIGURE 1-2. John Day Lock on Columbia River.

depth library of publications to capture the complex and diverse subject of inland navigation.

1.5 DESIGN PHILOSOPHY

The design for a modern inland waterway is to provide a navigation project that is safe, efficient, reliable, and cost effective with appropriate consideration of environmental and social aspects. Safety, efficiency, and reliability should be achieved before cost is optimized. Another goal of the design is to provide an environmentally compatible project where adverse impacts are avoided or minimized (McCartney 1986).

1.6 SAFETY

The safety of the waterway system relates to minimizing vessel collisions with other vessels, lock walls, and gates and other structural features such as bridge and port facilities. Vessel accidents create four problems; loss of life, the cost to repair the vessels and structures concerned, the possible closing of the waterway until the vessel is removed and the navigation structure is repaired, and potential oil spills or discharges of other hazardous material into the waterway. This can cause serious environmental harm and the waterway could be closed until cleanup is completed.

FIGURE 1-3. US Inland Waterway System.

1.7 EFFICIENCY

Efficiency of a waterway relates to the smooth uninterrupted flow of vessel traffic. The lock approaches must be designed to allow smooth lock entry and exit without delays. Port facilities need to be located a considerable distance from locks to avoid interference of port vessel movements and lock transit. Waiting areas for locks need to be close to the locks to speed vessel transit. Waiting areas also need to be located off channel so as to not interrupt through traffic. Locks need to be large enough to avoid the need to break up the tow which would require two lockages and additional time to break up and then reassemble the tow. The lock filling system needs to fill and empty rapidly. The gates need to open and close rapidly. And, if two or more locks are located at the same site, their layout should allow no interference of traffic when two locks are in use.

1.8 RELIABILITY

Reliability of a waterway system relates to the amount of time the system is able to pass traffic. Examples of reliability considerations are as follows:

a. Lock approaches should be kept ice and sediment free.
b. Lock gates should operate with a minimum of repair and down time.
c. Lock guide walls and chamber walls should be able to resist vessel impacts with no damage.

This page intentionally left blank

Chapter 2

PROJECT IDENTIFICATION

2.1 JUSTIFICATION

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

2.2 PRELIMINARY PLANNING

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

2.3 EVALUATION OF EXISTING STREAMS

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

2.4 COMMODITIES TO BE MOVED

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

2.5 FEATURES CONSIDERED

This background information can now be applied to the design of a waterway. The design procedure requires optimization of these interrelated features:

a. open river or canalization,
b. channel size,
c. tow size, and
d. if canalized, size and number of locks.

2.6 WATERWAY TYPES

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

2.7 OPEN RIVER

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

2.8 CANALIZED STREAMS

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

2.9 CANALS

Land-cut canals have been used to connect two bodies of water, to bypass rock outcrops and rapids, and to reduce the length or curvature of the navigable channel. Canals can parallel existing streams or continue overland to reach specific destinations. Construction of canals can be expensive depending on the amount and type of excavation, land acquisition, and availability of disposal areas. When connected to an existing stream or other body of water, locks might be required in the canal. In order to reduce the amount of excavation, canals might be routed through shallow lakes and estuaries. Stabilization structures might be required along the banks of the canal to reduce erosion of the banks due to waves created by traffic and wind. Canals tend to be narrow and shallow to minimize cost and could be affected by surges resulting from lock filling or emptying when relatively high-lift locks are used. Examples of land-cut canals are the Chain of Rocks Canal near St. Louis, MO, the New York State Barge Canal, the intracoastal canals, and the divide cut on the Tennessee–Tombigbee Waterway. The divide cut is discussed in Chapter 20 and shown in Figure 20-4.

2.10 BASIS OF SELECTION

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

2.11 COST ESTIMATES

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

2.12 BASIC PROJECT COMPONENTS

Navigation projects can be single purpose and only consider navigation, or a project may be developed for multipurposes such as flood control, hydropower, recreation, and water supply in addition to navigation. Therefore the basic components of a navigation dam could include:

a. spillway (gated or uncontrolled),
b. overflow embankment or weir,
c. nonoverflow embankment,
d. navigation pass,
e. lock or locks, and
f. outlet works.

2.13 SUPPLEMENTAL PROJECT COMPONENTS

The design of a single purpose or multipurpose project should accommodate each purpose as much as possible and develop a cost-effective functional plan. Common supplemental components are:

a. powerhouse,
b. fish passage facilities,
c. recreation facilities,
d. water supply intakes,
e. water quality, low-flow controls, and multilevel outlets, and
f. irrigation outlet works.

2.14 CHECKLIST FOR STUDIES REQUIRED

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

a. hydraulic and geological characteristics of the stream, and locations of existing bridges, highways, railroads, and industrial complexes;
b. channel depths and widths available and requirements for anticipated traffic;
c. need for channel realignment, training structures, and/or locks and dams;
d. optimum locations for locks, dams, and port facilities if required;
e. alignment and velocity of currents and movement of sediment in critical reaches and at proposed lock and dam sites;
f. effects of various arrangements of lock or locks, dam and overflow weirs, and auxiliary lock walls;
g. number and size of spillway gate bays and effects of overflow weirs and embankment on cost of project and on navigation conditions;
h. use of a navigable pass to reduce the height of lock walls;
i. effects of structures on flooding, overbank flow, and movement of sediment;
j. effects of various types of lock filling and emptying systems on navigation;
k. effects of developments on water quality and local environment;
l. feasibility of powerhouse installation and effects on navigation;
m. feasibility of water conservation methods;
n. effectiveness of various types of river training, stabilization structures, and navigation aids;

o. navigation and flow conditions during construction;
p. hydraulic model studies to determine:
   1. optimum design for spillway and stilling basin operating under various conditions (full or partial width);
   2. navigation conditions in lock approaches, best arrangement of locks, dam, and training structures, movement of ice and debris, and conditions during construction (comprehensive fixed bed or vessel simulator studies);
   3. effects of structures on movement of sediment, channel development in lock approaches and in critical reaches, and scour with various cofferdam plans for construction (comprehensive movable bed); and
   4. conditions at other locations as needed such as harbor entrances, port facilities, assembly areas, and at bridges (fixed or movable bed or vessel simulator studies);
q. baseline environmental and water quality data collection and evaluation, and consideration of applicable environmental laws and regulations;
r. relocations; and
s. historic and prehistoric preservation.

Chapter 3

PROJECT PARAMETERS

3.1 HYDROLOGY

3.1.1 General

Watershed hydrology is one of the first needs of developing a navigable waterway. The hydrologic conditions along the full waterway length will have an impact on the possible need for dams to establish reliable navigation. For instance, coastal regions, the Great Lakes, and the lower reaches of such major rivers as the Mississippi, Columbia, and Missouri are the only locations in the United States where existing depths or flows are adequate to maintain reliable navigation without dams. Hydrologic parameters also determine if the natural flows of the basin are adequate for continuous lock operations, or if supplemental supplies or special storage facilities will be required. Some navigation systems will traverse more than one river basin and require a hydrologic analysis of each basin. Basic hydrologic parameters for the design of all navigation dams are presented.

3.1.2 Basin Description

An understanding of certain physical features of a basin are necessary to properly evaluate the hydrologic and hydraulic functions. These physical features are needed to determine the rainfall-runoff and the discharge-stage relationships of the basin:

a. the location, size, shape, and general topographic nature of basins;
b. the names, drainage patterns, and longitudinal slopes of the mainstem and major tributaries;
c. the stream geometry including meandering patterns, channel widths, bank-line heights, cross-section shapes, bed slopes, and information on the historic changes to these features;

d. the density of vegetation cover over the basin and the soil types with respect to porosity and erodibility. An indication of watertable levels in that portion of the basin that could be affected by establishing permanent navigation pools;
e. the density of vegetation within the floodplain of the stream and the type and erodibility of materials compromising the bed and banks of the streams; and
f. all lake, reservoir, flood control, water supply, levee, irrigation, or other water resource projects that have caused modifications to streamflow discharges or durations. The dates when these modifications began affecting the natural flows need to be identified for proper correlation with streamflow records.

3.1.3 Hydrologic Data

The hydrologic studies for a river basin identify the discharges that a dam structure—located at any particular point within the basin—must be designed to control. Minimum, normal, and maximum discharges are all significant to the dam design. Furthermore, discharges must be determined that reflect not only existing basin conditions but also future basin conditions covering the economic life of the navigation system. For design purposes, stream discharges and stages at any site are commonly identified with respect to flow duration and exceedence frequency. The impacts of various flows on dam design are indicated.

3.1.3.1 Minimum Flows. These flows are essential to evaluate the quantity of water available for lock operations and for other potential project purposes such as water supply, low flow, hydropower, and the like. Minimum available flows will also identify the possible need for water storage or water import facilities to meet project purposes. At sites with limited water supplies, special seals may be proposed on spillway gates or other dam features to minimize water leakage.

3.1.3.2 Normal Flows. Moderate or commonly recurring flood flows are needed to establish the elevation of various project features such as access roads, lock walls, operating decks, and the like, and also project-related relocations and real estate requirements. Typical discharges used to determine the elevations include: the 2% duration flow, the 2-, 10-, 50-, and 100-year interval flood flows, and the standard project flood (SPF).

3.1.3.3 Maximum Flows. The maximum experienced flood of record is determined for each project, but the dam should generally be designed with adequate capacity to pass the probable maximum flood (PMF). Passage of this discharge may be exclusively through a gated spillway, but a portion could pass over the lock, the esplanade, and overflow weirs or embankments extending across the waterway overbanks.

Chouteau Lock and Dam is a typical navigation structure located on the McClellan–Kerr Arkansas River Navigation System. The pertinent discharge and stage data for this project are presented in Figure 3-1.

3.1.4 Hydrologic Data Sources

The records resulting from field measurements of both streamflows and climatological parameters such as rainfall, snowfall, evaporation rates, humidity, wind, and temperature are the basic source of needed hydrologic data. Streamflow records provide the simplest and most direct means of determining needed discharge data. However, streamflow recording stations are limited in number, often cover too short a time period, and occasionally are not reliable enough to provide all the flow information required for dam design. The normal procedure for obtaining the required supplemental data is to simulate flows from climatological data. In the United States, the sources of basic hydrologic data are as follows.

3.1.4.1 Streamflow Records. Most streamflow data within the United States are measured and recorded by the United States Geologic Survey (USGS) of the Department of the Interior. Occasionally, records are maintained by other agencies such as the Corps of Engineers, Soil Conservation Service, National Forest Service, various state agencies, and local municipalities. USGS records are published in convenient annual reports covering all gauges maintained within a specified state.

3.1.4.2 Climatological Records. In the United States, climatological data such as precipitation, evaporation, wind speed, temperature, and the like, are archived in various formats by the National Oceanic and Atmospheric Administration (NOAA), a unit of the US Department of Commerce. These data can be retrieved from annual reports or by magnetic tape from the NOAA data base. Most studies that have limited streamflow records utilize synthetic single storm events to determine flood frequencies. The general depth-area-duration rainfall data required for these computations are published by NOAA.

3.1.5 Hydrologic Model

For effective use in dam design, climatological data must be converted into streamflow data. This is normally accomplished by developing a math model to simulate the hydrologic response of the proposed project basin.

McClellan–Kerr Arkansas River Navigation System
CHOUTEAU LOCK AND DAM, VERDIGRIS RIVER, OKLAHOMA
From Design Memorandum No. 1, General Design

PERTINENT DATA

GENERAL	
Purpose of project	Navigation
Location of lock	3,400 feet east river mile 8.5
Location of dam, river mile	9.6
Upper pool elevation, feet	511.0
Normal lower pool elevation, feet	490.0
Minimum lower pool elevation, feet	487.0
STREAMFLOW AT DAM SITE, cfs	
Estimated maximum flood of record (1943)	224,000
Maximum modified flood of record	122,200
5-year recurrence interval flood, modified	50,000
10-year recurrence interval flood, modified	65,000
50-year recurrence interval flood, modified	126,000
Modified standard project flood	155,000
Discharge, 50 percent of time	620
Average flow	4,096
Minimum modified flow	230
Navigation design flood	65,000
Project design flood	155,000
Discharge, 2 percent of time	34,000
FLOOD DATA AT DAM SITE (TAILWATER ELEVATION, FEET)	
Estimated maximum river stage (1943)	529
Maximum modified flood of record	526.3
5-year recurrence interval stage, modified	515.8
10-year recurrence interval stage, modified	519.0
50-year recurrence interval stage, modified	526.6
Modified standard project flood	529.3
Discharge, 2 percent of time	510.7
Discharge, 50 percent of time	491.4
Average flow	496.0
Minimum modified flow	490.5
Navigation design flood	518.5
Project design flood	529.3

FIGURE 3-1. Pertinent Hydrologic Data for a Typical Navigation Dam Project.

3.1.6 Flow Computations

Establishing a navigation system through a basin will usually affect the hydrology of the basin. Consequently, both existing and postproject conditions must be determined. Basic hydrologic computations required for all studies include the following.

3.1.6.1 Probable Maximum Flood (PMF). This hypothetical event represents the flood resulting from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible to occur in a region. The National Weather Service has identified the Probable Maximum Storm (PMS) upon which the PMF is based for all regions of the US. The precipitation data for these storms are contained in a series of regionally oriented Hydrometeorological Reports (HMRs).

3.1.6.2 Standard Project Flood (SPF). The SPF is runoff resulting from the Standard Project Storm (SPS)—the rainfall representing the most severe storm that is considered reasonably characteristic of the region in which the drainage basin is located. For very large watersheds, the SPS is frequently estimated to be half of the PMS as previously determined.

3.1.6.3 Flood Frequencies. The designs of many dam features are based on the frequency of floods at the project site. Flood frequencies are identified as the time in years between which a particular flood discharge is likely to recur. For instance, a 50-year recurrence interval flood discharge would have an average time interval of 50 years between occurrence of a given or greater magnitude discharge. It would have a 2% chance of being equaled or exceeded in any 1 year.

3.1.6.4 Flow Duration. Lesser project flows are commonly expressed with respect to their duration—the percentage of time that a particular discharge is equaled or exceeded. Discharge-duration curves are determined from the total period of flow data records. These records are also used to determine existing minimum flow conditions.

3.2 HYDRAULICS

3.2.1 General

Hydraulic studies for navigation dam design generally cover two distinct phases. One phase is establishing the stage-discharge relationship over the entire area affected by the proposed project under both existing and post-project conditions. The studies in this phase coordinate closely with the hydrologic studies. They establish stage-discharge relationships both at specific sites and over extended river or channel reaches affected by the project. The second phase of hydraulic studies involves the design of dams and other structures—their type, shape, size, and siting to ensure satisfactory hydraulic performance. The required stage-discharge studies presented here cover channel discharge rating curves, water-surface profiles, and establishment of navigation pool elevations.

3.2.2 Channel Discharge Rating Curves

Stage-discharge relationships are required to initiate water-surface profile computations and also to establish the tailwater conditions for the hydraulic design of dams and their spillway structures. The basic source of discharge rating curves is stage-discharge records collected at stream-gauging stations. These stations are located at relatively fixed stream cross-sections such as bridge openings or weir structures where the stage-discharge relationships stay relatively fixed with time. Most existing stream-gauging stations are established and operated by the USGS. However, existing station locations are limited and establishing new stations for specific projects may be advisable.

3.2.2.1 Stream Changes. Existing rating curves can be determined directly from stream records. However, these curves are affected by project-related changes to the downstream channel alignments, cross-sections, channel stabilization measures, and established navigation pools. Postproject or future condition rating curves must reflect these changes. The effects of these changes can only be estimated until the project design has been finalized, so the postproject rating curves are generally adjusted and refined throughout the project design process.

3.2.2.2 Backwater Effects. Occasionally, channel rating curves under both existing and postproject conditions are affected by backwater effects from downstream receiving rivers, major tributaries, lakes, or bays. In such instances, channel stages cannot be related to a specific stream discharge. For any specific channel discharge, the water level would vary over a range of stages depending on the downstream backwater stages. The specific rating curve application will determine if a low, high, or average backwater stage should be considered. A study of experienced coincident discharge and stage conditions can be helpful in selecting appropriate backwater conditions.

3.2.3 Water-Surface Profiles

A key tool in the development of a navigation system through a drainage basin is the model used to calculate water-surface profiles for both existing and postproject conditions. By comparing the two profiles over a wide range of the discharges, the hydraulic impacts of establishing various dam locations and navigation pool elevations can be evaluated. The preproject and postproject profiles are also needed to evaluate the effects on flood heights, relocation requirements within the pool length, and flooding effects on adjacent real estate.

3.2.3.1 Computation Procedures. Navigation projects are located on or along streams that flow within the subcritical range. Development of a basin-specific computer model for calculating standard backwater computations is the normal method for determining water-surface profiles.

3.2.3.2 Multiple Computations. During floods when water levels are well over the riverbanks, flow patterns can become very complex. Manmade obstructions such as transportation embankments, levees, building developments, dams, and the like, or even natural features such as swales, cutoffs, or divided channels can require multiple backwater runs through the study reach to properly identify water profiles. In some complex study reaches, development of a hydrodynamic (unsteady or multidimensional) math model as an alternative to standard backwater computations may be advisable.

3.2.3.3 Profile Plots. Plotting existing and postproject water-surface profiles over a complete navigation system can be a major undertaking.

3.2.4 Specific Profile Uses

Following are descriptions of some of the most common uses of water-surface profiles in navigation dam design.

3.2.4.1 Real Estate. The extent of lands acquired under fee-simple purchase or under easement rights purchase are based on envelope curves that directly compare preproject with postproject water-surface profiles.

3.2.4.2 Relocations. Highway and railroad embankments, bridges, overhead utility crossings, flood protection levees, drainage structures, and a multitude of riverside facilities such as water and sewage treatment plants, pumping stations, parks, and industrial and residential areas are all affected by floodwaters when a navigation pool is established. Alteration, protection, or relocation of all these facilities are based on the water-surface profiles.

3.2.4.3 Lock and Dam Features. The elevations of a number of structural features are determined from water-surface profiles. For example, on the Arkansas River navigation system the channel was anticipated to be navigable for flows up to the 10-year recurrence interval flood. Flow velocities at larger floods were expected to be too high for safe or efficient operation of most tows. Consequently, the top of lock walls and the esplanade areas were set at the higher of 10 ft above the navigation pool or 2 ft above the 10-year recurrence interval flood. Access roads were set at the 10-year recurrence interval flood. Other feature elevations were similarly dependent on the profiles.

3.2.4.4 Groundwater Table. Permanently establishing navigation pools at elevations near the top of riverbanks may cause significant changes to the water table levels on adjacent lands. Saturated soils can adversely affect or destroy the productivity of farmlands. Established land drainage facilities can lose their efficiency by reduction of the hydraulic heads between the fields or ditches and the river. A special study of watertable changes resulting from proposed navigation pools may be necessary. Such a study was conducted by the USGS along reaches of the Arkansas River. The study included an inventory of over 1,500 existing wells, installing and periodic reading of an additional 1,500 wells, 27 pumping tests, numerous aquifer sample tests, and geologic mapping. The study covered the affected lands in each proposed pool. It led to shifting of some project sites to minimize adverse drainage problems.

3.2.5 Navigation Pool Level Stability

In addition to the flooding impacts of an established navigation pool elevation, consideration should be given to the operational stability of the selected pool. A navigation dam should provide a fixed pool elevation with as little stage variation as possible. Attainment of this goal best promotes reliability and growth in waterway traffic and also simplifies development of port facilities. A number of factors that have an effect on pool stability need to be considered.

3.2.5.1 Project Purposes. Pool stability for any navigation dam can best be maintained by eliminating or minimizing those project purposes that require water storage within navigation pools. To the extent possible, project purposes requiring storage should be located in headwater or tributary projects to the navigation channel. If included in navigation dams, the water requirements should be restricted to amounts less than the minimum inflows into each pool minus that amount required for navigation lockage and dam leakage. Recreation purposes normally are enhanced by stable pools. Many navigation pools include hydroelectric power plants. To minimize pool fluctuations, they should be operated as run-of-river plants with allowable pool fluctuations limited to three feet or less. Allowable tailwater fluctuations should be established. Rates of change in pool and tailwater elevations should also be considered.

3.2.5.2 Dam Head. Stable navigation pools are more easily maintained with high-head rather than low-head dams. This is because high pools are less frequently affected by flood stages, particularly in the downstream portion of the pools. However, existing developments are so extensive in many reaches of those rivers that can economically justify navigation projects that low-head dams with pool levels contained within the riverbanks are usually mandated. In such instances, stable pools can best be maintained with dams that have high capacity spillways that minimize upper and lower pool head differentials during flood conditions. Both high- and low-head dams are common on navigation projects located throughout the United States.

3.3 SEDIMENTATION

3.3.1 General

Sedimentation problems should be grouped into two main categories: local scour and deposition problems, and general degradation and aggradation problems. The first are controlled or influenced primarily by the hydraulic design of the project structures. The second are the result of the stream's response to changes in the discharge hydrograph and sediment transport caused by the proposed navigation projects.

3.3.2 Problems

Alluvial rivers tend to establish an equilibrium between the water and sediment loads imposed upon them. Any significant modifications to the system (realignments, locks, dams, etc.) will disrupt this balance and a period of adjustment will occur as the stream attempts to reestablish a new state of equilibrium. During this period of adjustment, sediment-related problems are increased. Development of a river system for navigation involves the construction of several major work components such as locks and dams, bank stabilization, reservoirs, and realignments. The impacts of each of these components of work can be assessed individually. However, the ultimate response depends on how the system integrates these individual impacts in an effort to attain a new equilibrium state. Because of this complexity it is difficult and sometimes dangerous to develop definite rules or trends that apply to all navigation projects. Design criteria and techniques that have been successful on one river system may not be feasible on another system which has different hydrologic or geomorphic characteristics.

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

3.3.3 Sediment Data Needs

Knowledge of sediment transport, in terms of both quantity and quality, is essential for design of river engineering works on alluvial streams. The primary sediment problems associated with navigation systems are related to deposition in navigation pools, degradation below dams, and streambank erosion. In order to assess these problems, certain basic data must be available. These basic data should include suspended sediment samples, bed-load samples (if possible), bed material samples, and borings in the streambed and banks. Sampling stations should not be restricted to the limits of the navigation project but should include upstream and downstream reaches, as well as major tributaries.

3.3.4 Sedimentation Study

Potential sediment problems may be minimized and in some cases eliminated by conducting a detailed sedimentation study of the entire stream system. As one component of a comprehensive geomorphic analysis, the sedimentation study is aimed at developing an improved understanding of the significant sedimentation processes within the basin. The major emphasis of this type of study should be on analyzing the channel morphology and sedimentation phenomenon during the historic period, although long-term system changes are also considered. As a minimum the sedimentation study should document the variations in sediment transport (size and quantity), identify all major sources of sediments (bed and banks, tributaries, etc.), locate degrading, aggrading, and stable reaches, and establish the range of flows transporting the majority of sediments. Correlating the results of the sedimentation study with historical changes in the basin (channel improvements, land use, reservoirs, etc.) enables the engineer to develop a firm understanding of past and present sedimentation processes. With this information the effects of anticipated project features can be analyzed qualitatively. A qualitative analysis of this nature is essential for the development and interpretation of results from sediment transport models.

3.3.5 Analysis Tools

A number of methods are available to design engineers to analyze sedimentation problems associated with the design and operation of navigation projects. These tools include numerical models, physical models, and analysis of prototype data. Prior to the use of any of these tools, the designer should have developed an understanding of the existing sediment regime of the planned navigation system. The methods for establishing baseline sediment study were discussed in Section 3.3.4. Also prior to the development of either numerical or physical models, the designer should have a knowledge of the expected sedimentation changes as a result of altering the river system. This knowledge should help the design engineer in the selection of the model to be used, study limits for the model, and estimation of the cost of the model study. The first analysis tool used by the engineer designing the navigation projects should be review of sedimentation control methods that have been used on other navigation projects. Sediment control measures have been used on a number of rivers in the US including the Missouri, Ohio, Mississippi, Arkansas, Ouachita, Red, and Black Warrior Rivers. A review of what has worked and more important what has not worked as a means of controlling and reducing sediment problems on these rivers will provide the designer of a new navigation system with a basis for developing solutions to sediment problems that develop during the model studies. The following tools are available for the detailed studies. It should be emphasized that the tools listed, whether they be numerical or physical in character, have all been successfully applied to navigation sedimentation problems and if correctly applied using good engineering judgment will provide reliable guidance in selections of sediment control measures.

Mathematical models are the first tool to consider for analysis of sedimentation. These models can be one-, two-, or three-dimensional. They usually require the input from a river flow model for the river current parameters.

Before beginning the detailed design of a proposed navigation project, a movable-bed physical model study should be considered. The cost of the model study is small when compared to the total engineering design and construction cost of a navigation project, and the results of a physical model study are often useful in verifying the design developed in numerical model studies and in providing guidance for design of the overall project. Each lock and dam should be physically modeled with a movable bed prior to detailed design; if the project requires major channel realignment a typical reach model should also be considered.

3.3.6 Sediment Control Measures

A number of methods for controlling sediment problems are associated with navigation projects. These methods of sediment control involve the management of sediment problems at an isolated location, and source reduction of sediment either by bank stabilization or an upstream reservoir. Control of sediment problems at isolated locations involves such things as dikes, bank stabilization, and structural modifications to the lock and dam.

Controlling the source of sediment must be carefully analyzed to ensure that the control does not have adverse impacts upstream or downstream of the project. The reduction of the upstream sediment source does not in itself imply overall reduction of sediment problems. In areas where no sediment source is obvious, measures such as covering the sediment source with polyethylene filter cloth should be considered. When considering an upstream reservoir as a method for reducing sediment inflows, the need for grade control in the channel downstream of the reservoir should also be considered. This review of grade control structures should also include tributaries to main channels that might be subject to degradation resulting from the construction of upstream reservoirs.

3.4 ICE CONDITIONS

3.4.1 General

The prediction of extent and duration of ice conditions at navigation dams is necessary to allow development of ice control methods. The extent of ice problems can be determined by review of historical records and monitoring the site conditions during the study.

3.5 SOURCE

The majority of this chapter was extracted from the US Army Corps of Engineers Manual, EM 1110-2-1605, *Hydraulic Design of Navigation Dams.*

Chapter 4

WATERWAY TRAFFIC

4.1 GENERAL

The development of inland waterways for navigation usually can only be justified on the basis of commercial traffic. Therefore, design of the waterway should consider the types of equipment that would be using the waterway and their principal characteristics. River commerce in the United States is handled chiefly by barge tows consisting of a towboat pushing one or more barges, depending on the characteristics of the waterway, facilities provided, type of cargo, and size and power of the towboat. The tow speed and direction are controlled by the towboat, which is at the stern; the head of the tow is at the other end, sometimes from one barge length to more than 1,200 feet away. Towboats vary in size, power, and maneuverability and, therefore, in their capability of handling loads under various conditions. Figure 4-1 indicates some of the equipment in more general use in the United States in 1979. A new tug built for the Columbia/Snake waterway system has the following statistics.

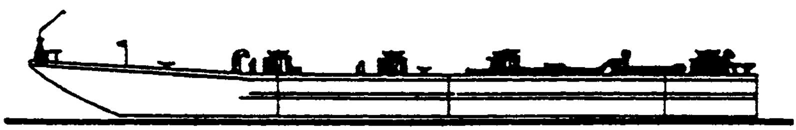
Name:	Deschutes
Cost:	$3.8 million
Builder:	J.M. Martinac, Tacoma, WA
Length:	93 ft
Beam:	36 ft
Water:	requires 12 ft
Crew:	two, a pilot and a deckhand
Power:	twin Detroit diesel engines rated at 1,600 horsepower each at sustained use, driving two 90-inch propellers in swiveling nozzles
Duties:	grain tows from Idaho, Columbia River ship assistance calls
Christening:	11 July 1997.

OPEN HOPPER BARGES

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

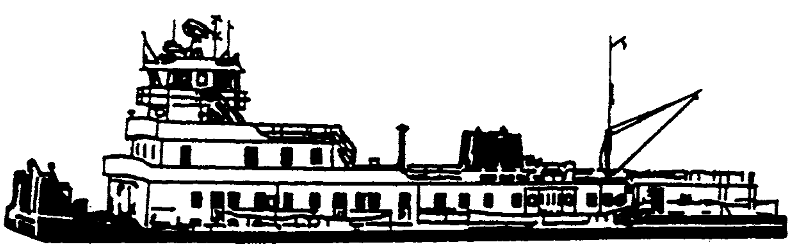
COVERED HOPPER BARGES

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



INTEGRATED CHEMICAL AND PETROLEUM BARGES

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



TOWBOATS

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

FIGURE 4-1. Predominant Barge and Tow Types.

Photos of tows and tow currently in use are shown in Figures 4-2 through 4-6. The US integrated barge and tow configurations differ from the European inland transport vessels which are usually single-unit, self-propelled cargo carriers. Figures 4-7 and 4-8 show typical cargo vessels used on European waterways.

4.2 TOWBOAT CONTROLS

The towboat pilot is usually a considerable distance from the head of the tow, and the only means of control of the tow(s) is the action of the towboat rudder and propeller screws. The pilot's control of the tow depends on the

FIGURE 4-2. Typical Tugboat on Lower Mississippi.

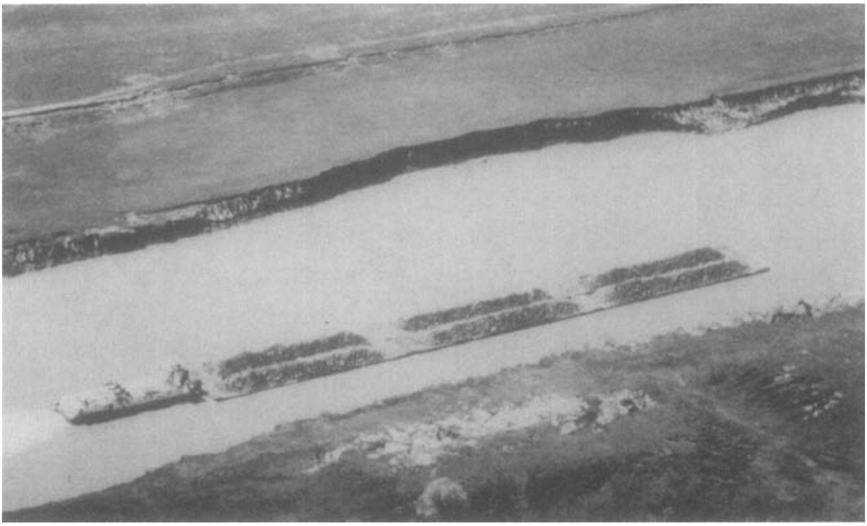


FIGURE 4-3. Tow on Red River, LA.

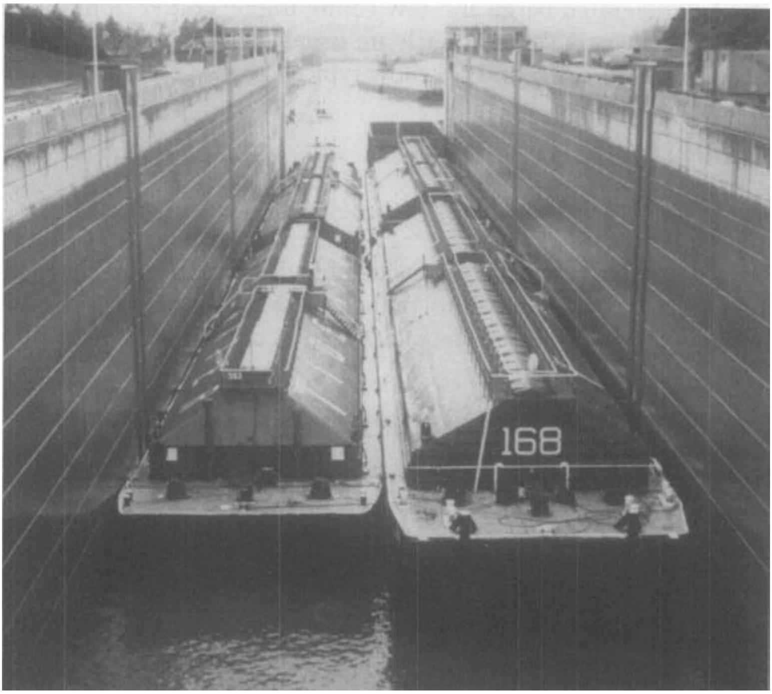


FIGURE 4-4. Tow Entering Bonneville Lock on the Columbia River.

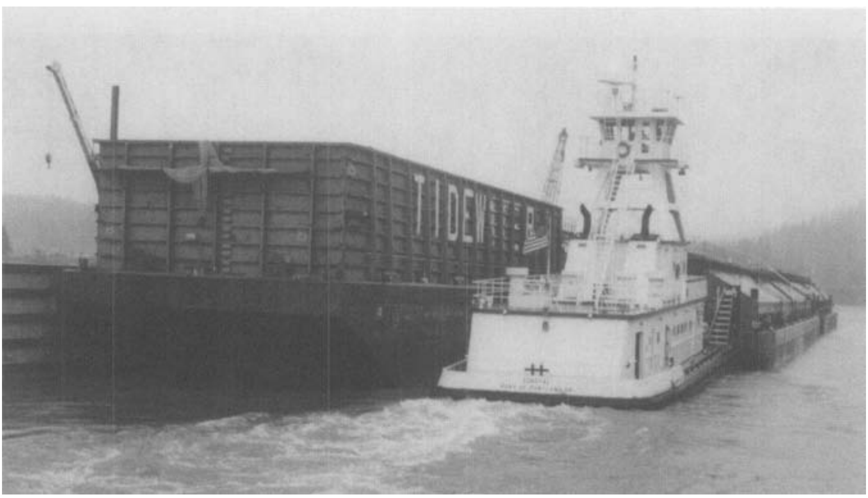


FIGURE 4-5. Tow Leaving Bonneville Lock on the Columbia River.

FIGURE 4-6. Tow Entering Bay Springs Lock on the Tennessee–Tombigbee Waterway.

maneuverability and power of the towboat, and the ability to anticipate the effects of currents, navigation aids provided, and visibility. The power of the towboat and the action of the rudder affect the movement of the tow, as do the direction and velocity of currents, wind, ice, drift, and channel dimensions. The towboat rudder or rudders develop a side thrust when placed at an angle to the direction of flow. This thrust is proportional to the area of the rudder affecting the currents, the angle of the rudder to the currents, and the square of the velocity of the currents directed against the rudder by the propeller in relation to the speed of the towboat. When a towboat is reducing speed in relation to the velocity of currents, it is losing rudder power; and when moving in the same direction and at the speed of the currents, the tow has no rudder control. When a towboat changes directions, the action of the rudder moves the stern of a forward-moving tow in a direction opposite to that of the turn. The pivot point of the turn from a standing position in slack water is some distance forward of the midpoint (about 30% of its length from the head of the tow). When the tow is underway and proceeding ahead, the pivot point moves forward and could be some distance beyond the head of the tow depending on the speed of the tow and the direction of the currents in relation to that of the tow. This explains why the stern of a

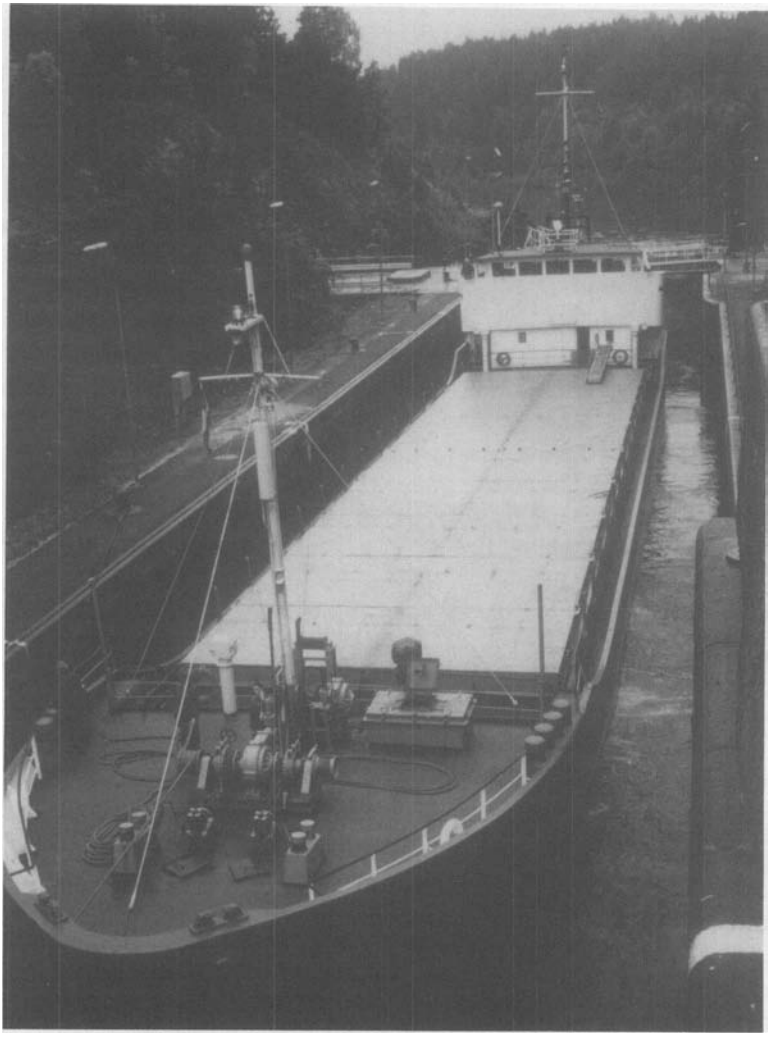


FIGURE 4-7. Cargo Vessel in Trollhatte Canal Locks, Sweden.

tow will not necessarily follow the same path as the head when turning, going around bends, or attempting to compensate for adverse currents.

4.3 MANEUVERABILITY OF TOWS

The maneuverability of towboats varies depending on the size and number of rudders, power versus load of the towboat, and special equipment, such as Kort nozzles or bow steerers. Most towboats are equipped with flank-

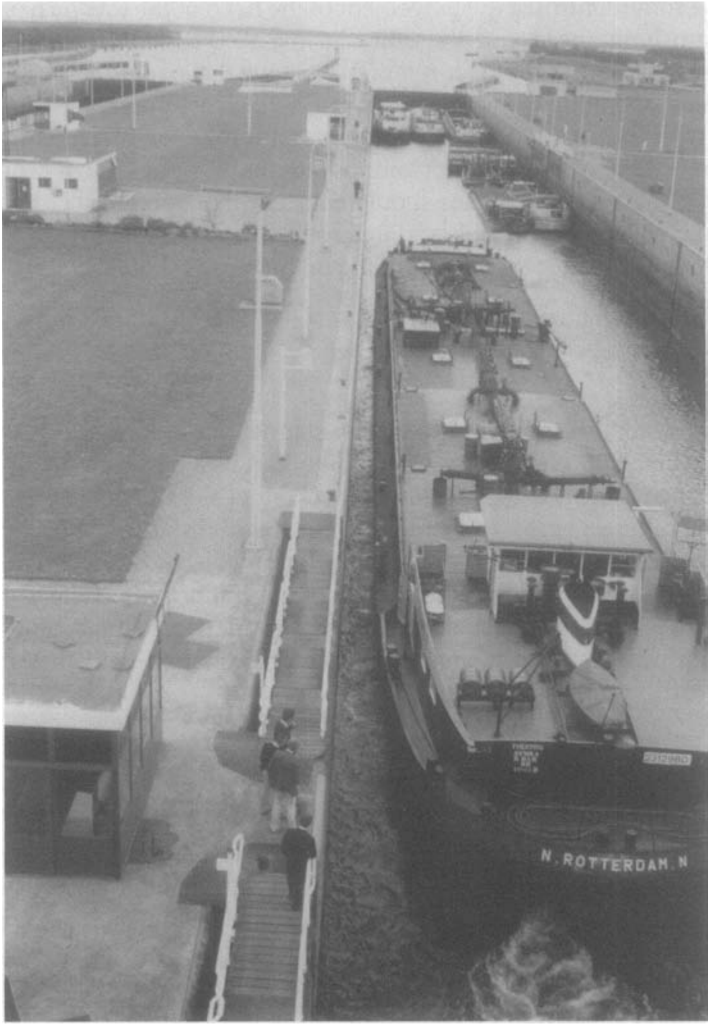


FIGURE 4-8. Cargo Vessel in the Netherlands.

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

4.4 VISIBILITY

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

4.5 EFFECTS OF CURRENTS

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

4.6 SOURCE

This chapter was extracted from the US Army Corps of Engineers Manual, EM 1110-2-1611, *Layout and Design of Shallow-Draft Waterways*. Additional information and examples can be found in this Manual.

Chapter 5
CHANNEL SIZE AND ALIGNMENT

5.1 GENERAL

5.1.1 Channel Characteristics

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

5.1.2 Channel Dimensions

Section 5 of the Rivers and Harbors Appropriation Act approved 4 March 1915 (PL 291,1915), outlines the basis for channel dimensions as follows.

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

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

5.1.3 Channel Requirements

Channel dimensions required for navigation will depend on channel alignment, size of tow, and whether one-way or two-way traffic is to be accommodated. One-way traffic can be justified on segments of some waterways when there is a low volume of traffic, passing lanes are provided on long reaches, and close traffic control and communication are maintained. An example is the Verdigris River portion of the McClellan–Kerr navigation system. The one-way channel segments are 9 ft deep by 150 ft wide. Passing basins of 100 by 700 ft are provided at varying distances, but average about 1 every 5 miles. The towing industry regulates itself on the use of passing basins and has not voiced any strong complaints. Providing for two-way unrestricted traffic would generally result in a safer and more efficient waterway. Channel dimensions and alignment provided can affect construction and maintenance cost and the development of traffic on the waterway. Providing a straight channel can reduce the length of the waterway and require less channel width than with a sinuous channel. However, channels in natural streams tend to meander with most of the length of the low-water channel occurring in bends of various curvature. In streams carrying sediment, long straight reaches would tend to be unstable and difficult to maintain.

5.2 CHANNEL DESIGN

5.2.1 Channel Cross-Section

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

5.3 CHANNEL IN STRAIGHT REACHES

5.3.1 Minimum Width

The minimum channel widths required for safe navigation in straight reaches depend on the type and size of equipment in general use on the waterway, alignment and velocity of currents, intensity of the prevailing wind, how well the channel limits are defined, navigation aids provided, and whether one-way or two-way traffic is permitted. The minimum channel width should provide for the width occupied by the tow, clearance between the tow and channel limits, and clearance between tows for two-way traffic. Operating experience has indicated that the minimum clearance required for reasonably safe navigation in straight reaches should be at least 20 ft between tow and channel limits for two-way traffic, 40 ft for one-way traffic, and at least 50 ft between tows when passing. When structures or mooring areas that could constitute a hazard are located along the channel limit line, greater clearances should be provided. Also, additional clearance should be provided in channels with restricted cross-sectional area or where adverse currents would be encountered. Because of the larger cross-section of channels designed for two-way traffic where passing occurs in a relatively short reach, the clearance required between tows and channel limit is less than for channels designed for one-way traffic. As a guide, the minimum channel widths required for tows of various sizes are shown in Table 5.1.

5.3.2 Minimum Crossing Distance

Crossings are straight reaches between alternate bends and are common in meandering streams. Tows leaving one bend, usually from along the concave bank, have to cross toward the opposite bank to approach the concave bank of the next alternate bend. The distance required for a downbound

TABLE 5-1

	Channel Width (Ft)	
Tow Width (Ft)	Two-Way Traffic	One-Way Traffic
105	300	185
70	230	150
50	190	130*

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

tow to make the crossing without flanking or excessive maneuvering will depend on the size of tow, the width of channel, and the alignment and velocity of currents. The average length of channel required for various sized tows to cross from one side of the channel to the opposite side under varying conditions as determined from the results of model studies is shown in Figure 5-1. These results are based on moving minimum-powered tows located adjacent and parallel to one side of the channel crossing to the opposite side with currents generally parallel to the bank lines. In most crossings, currents will tend to move from along the concave bank of one bend toward the concave bank of the next bend downstream, particularly during the lower flows. In such cases, tows can make the crossing in a shorter distance than indicated. Also, tows with greater power and controllability or tows flanking can make the crossing in a somewhat shorter length of channel than that indicated by the results of the model study. Although not exact, the information in Figure 5-1 provides a good general indication of the length of straight reach that should be provided between alternate bends, particularly when short radius bends and limited channel widths are involved.

5.4 CHANNEL WIDTHS IN BENDS

5.4.1 Orientation of Tows in Bends

The need for additional widths in bends has been known and recognized but little or no information has been available on the amount required. Model studies have provided a basis for computing channel widths occupied by tows of various sizes navigating in bends of different curvature. Figure 5-2 shows one of these navigation model runs in bendways. In making a turn, the stern of a tow is moved laterally in a direction opposite to the direction of the turn. In negotiating a bend, the tow assumes and maintains an angle to the channel alignment which is referred to as the deflection angle (also referred to as the drift angle). The width of the channel required is a direct function of the deflection angle assumed by the tow and the length and width of the tow (Figure 5-2). The deflection angle assumed by a tow is dependent on many factors, the most important of which are radius of bend, size of tow, and the length of the bend up to about 90 degrees. Other factors affecting the deflection angle include current alignment and velocity, speed of tow with respect to that of the currents, draft of the tow with respect to channel depth, direction of travel (upstream or downstream), tow driving or flanking when downbound, and alignment and position of tow entering the bend. As a general rule, the deflection angle assumed by a tow increases rapidly as the bend length decreases to less than about 4 or 5 times the length of the tow.

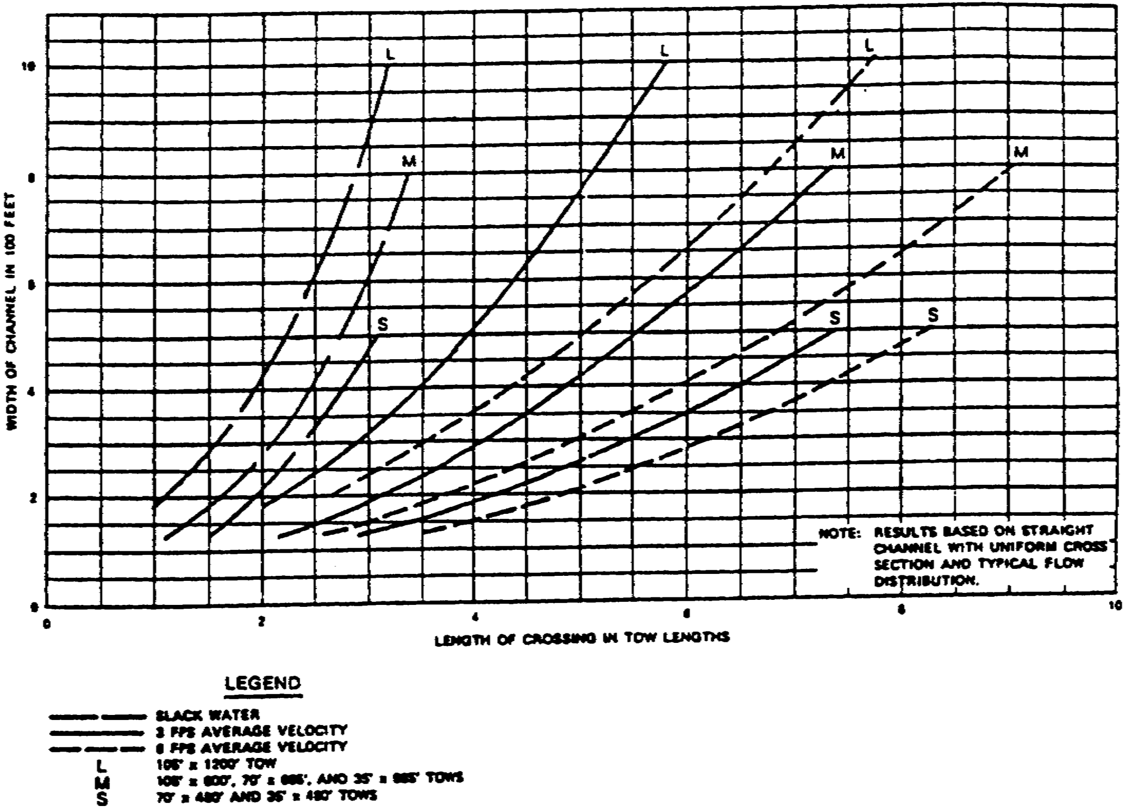


FIGURE 5-1. Length of Channel Required Between Alternate Bends.

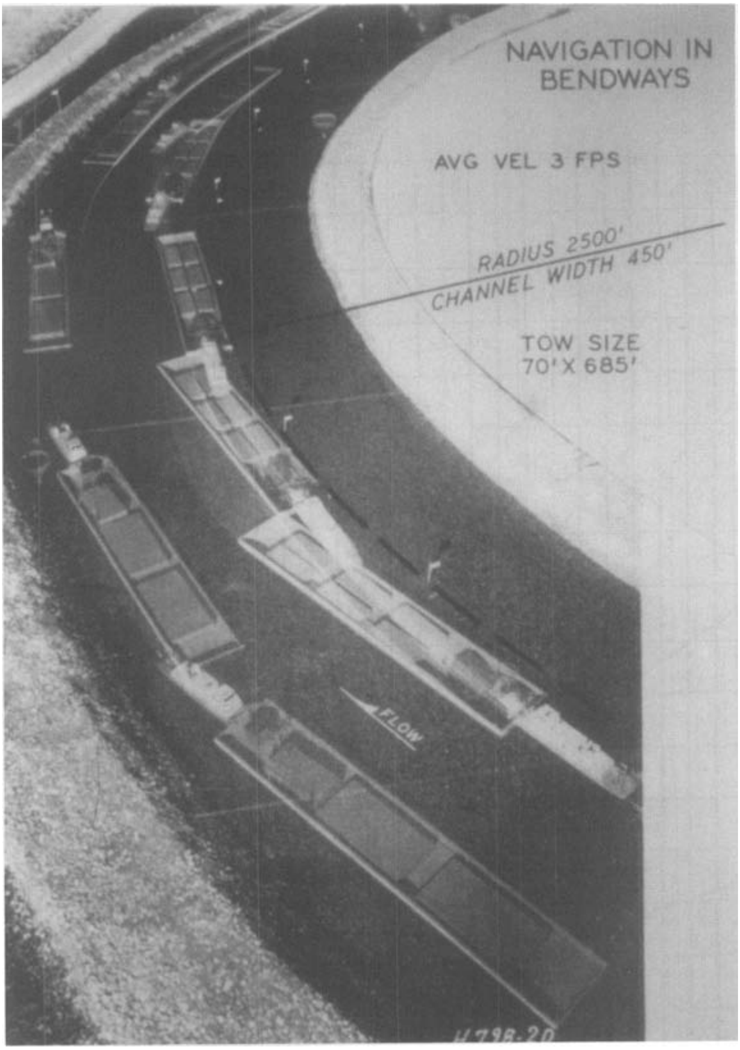


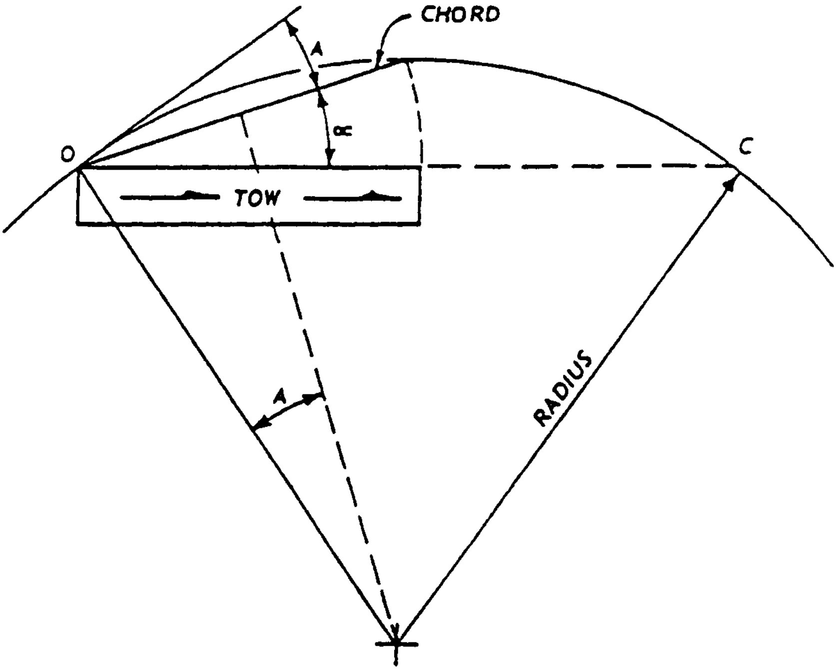
FIGURE 5-2. Navigation in Bendways Model Test.

5.4.2 Determining Channel Widths Required in Bends

If the deflection angle (see Figure 5-3) assumed by a tow is known, a reasonably accurate channel width required can be determined from one of the following equations,

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

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



LEGEND

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

FIGURE 5-3. Description of Deflection Angle α.

where

CW_1 = channel width required for one-way traffic, ft
CW_2 = channel width required for two-way traffic, ft
α_d = maximum deflection angle of a downbound tow, degrees
α_u = maximum deflection angle of an upbound tow, degrees
L = length of tow, ft
W = width of tow, ft
C = clearance required between tow and channel limit for safe navigation, ft
C_t = minimum clearance required between passing tows for safe two-way navigation, ft

Figure 5-4 shows the swept path of various size tows for a 3,000 ft radius channel bend.

5.4.3 Deflection Angles

The deflection angle assumed by different size tows in bends of various curvature based on results of model studies are shown in Figures 5-5 to 5-10.

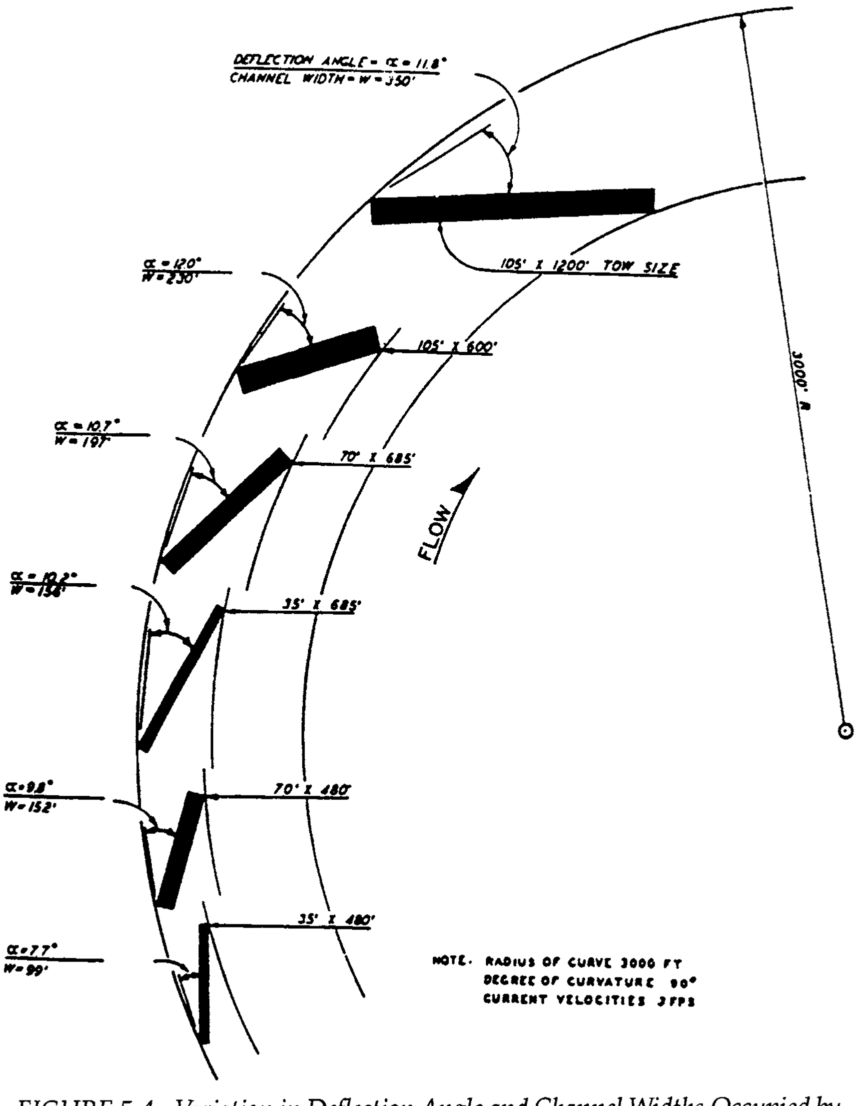


FIGURE 5-4. Variation in Deflection Angle and Channel Widths Occupied by Tows of Different Sizes.

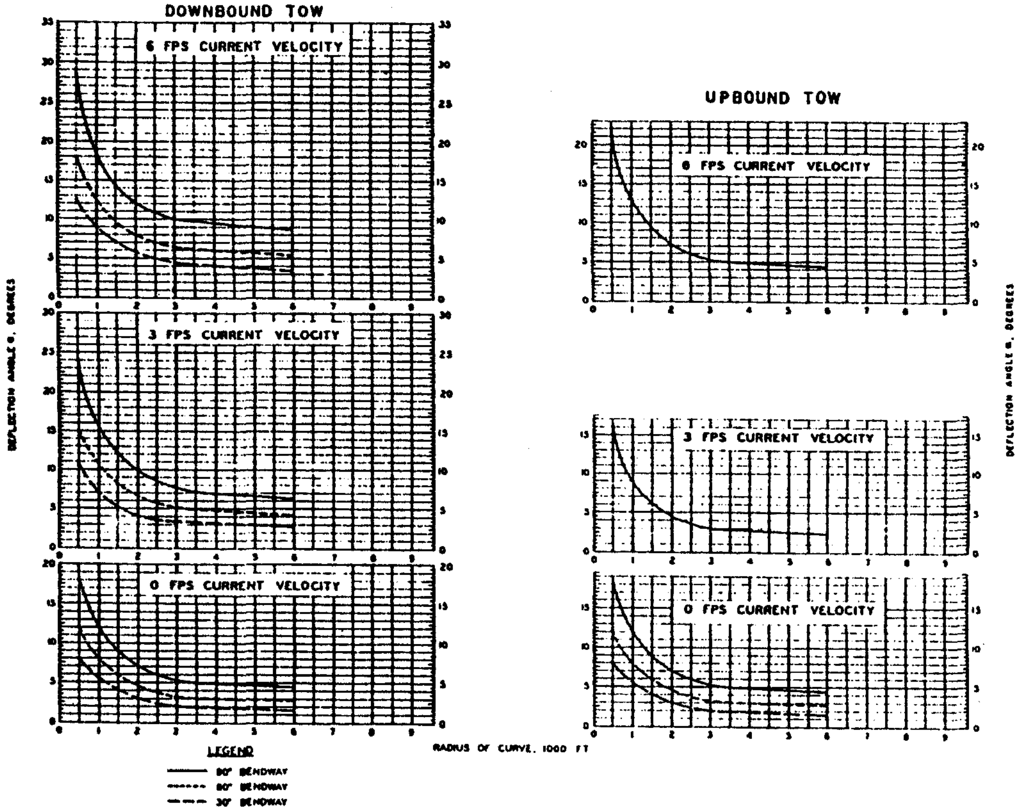


FIGURE 5-5. *Deflection Angle for Tows Driving Through Bends Forming Uniform Curves: Tow Size: 35 ft wide by 480 ft long, submerged 8 ft.*

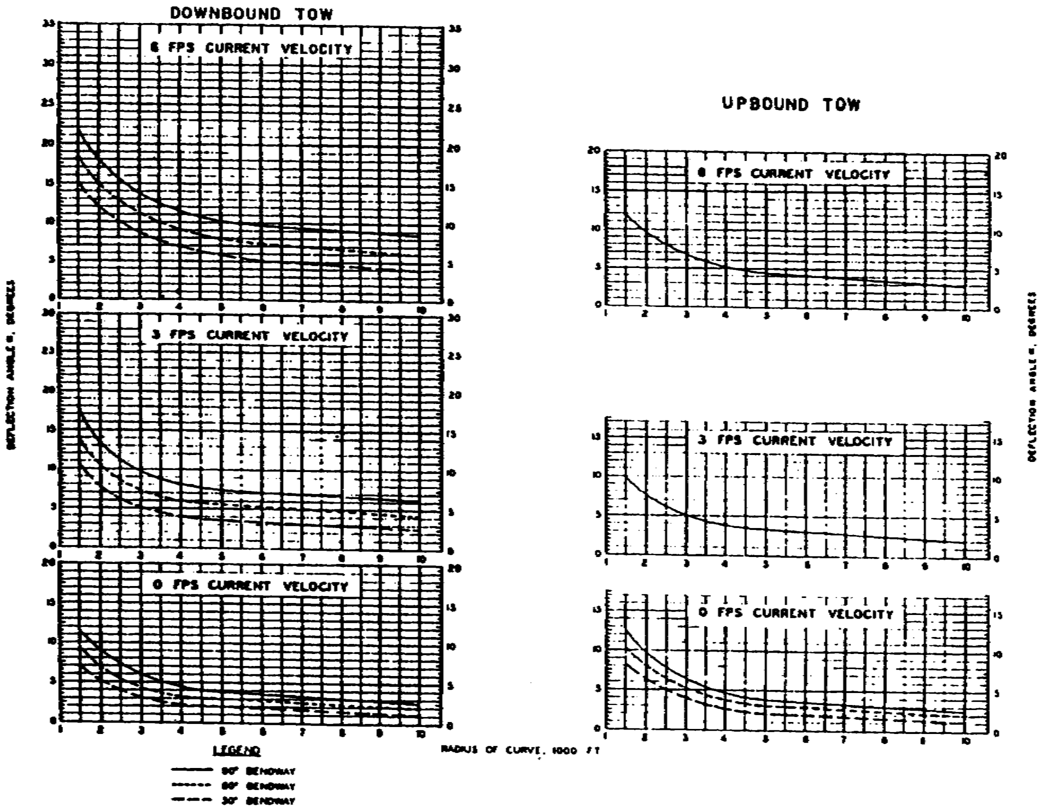


FIGURE 5-6. *Deflection Angle for Tows Driving Through Bends Forming Uniform Curves: Tow Size: 35 ft wide by 685 ft long, submerged 8 ft.*

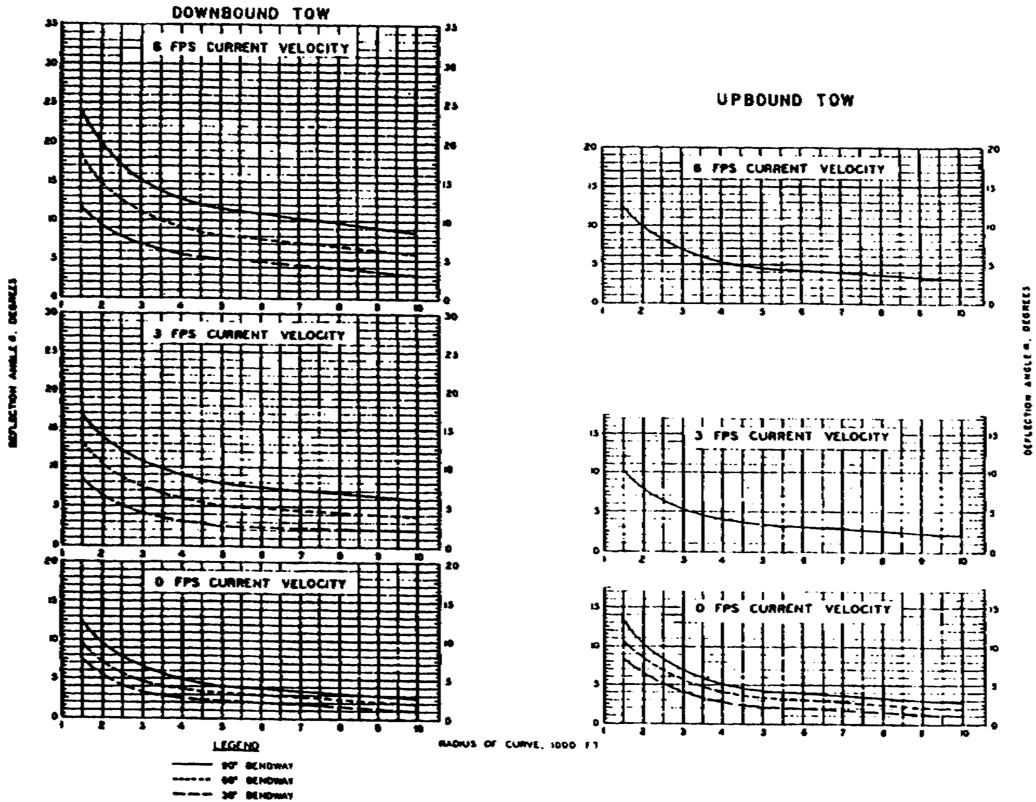


FIGURE 5-7. Deflection Angle for Tows Driving Through Bends Forming Uniform Curves: Tow Size: 70 ft wide by 685 ft long, submerged 8 ft.

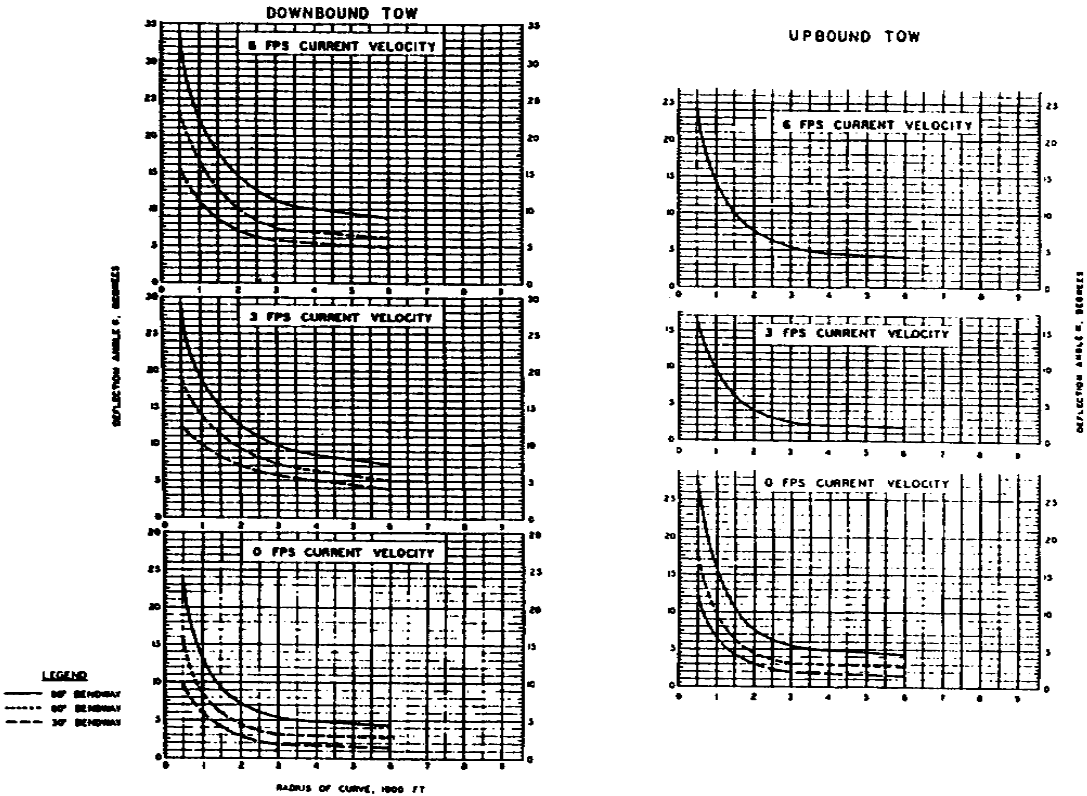


FIGURE 5-8. Deflection Angle for Tows Driving Through Bends Forming Uniform Curves: Tow Size: 70 ft wide by 480 ft long, submerged 8 ft.

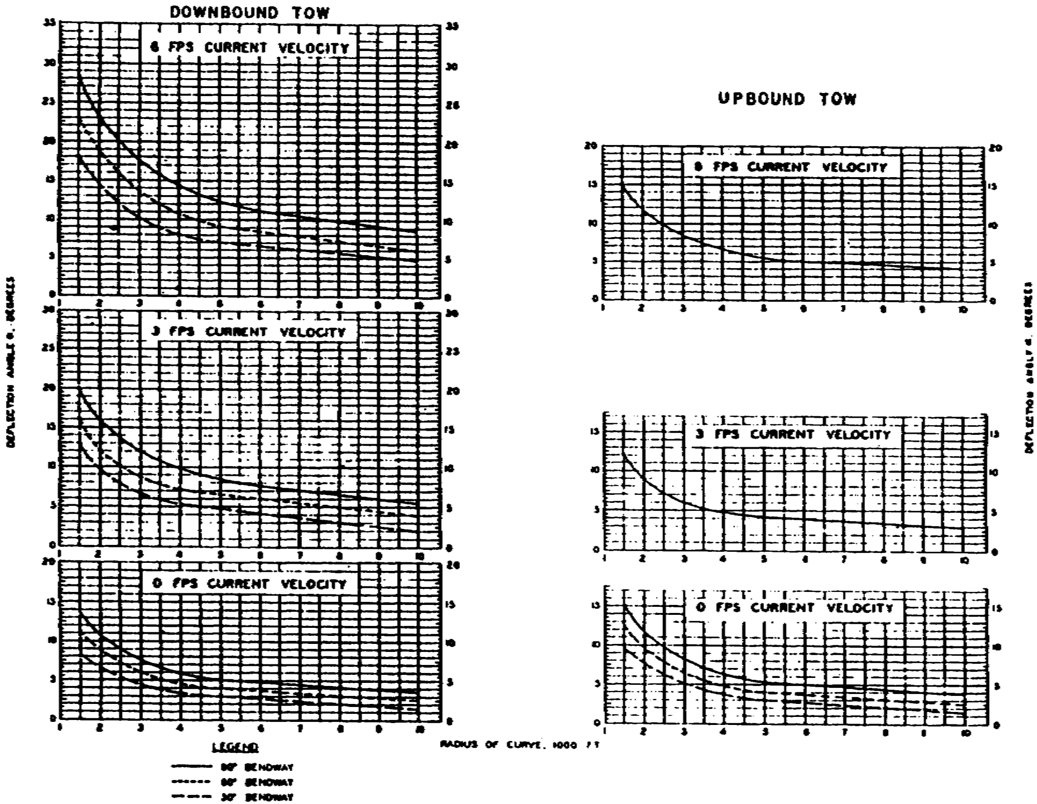


FIGURE 5-9. Deflection Angle for Tows Driving Through Bends Forming Uniform Curves: Tow Size: 105 ft wide by 600 ft long, submerged 8 ft.

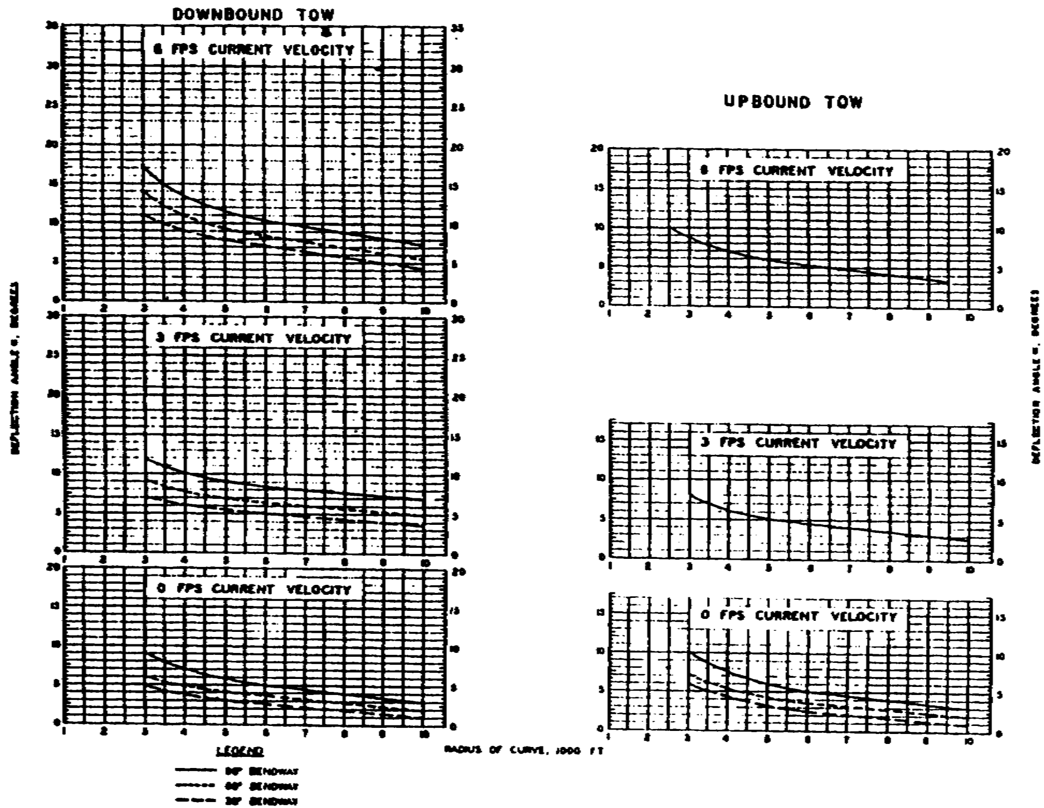


FIGURE 5-10. Deflection Angle for Tows Driving Through Bends Forming Uniform Curves: Tow Size: 105 ft wide by 1,200 ft long, submerged 8 ft.

These results are based on tests with tows having the minimum power required to navigate the waterway under the conditions indicated. Results for downbound tows are based on driving the bend. Channel dimensions for downbound tows flanking can be approximated by using the deflection angle for an upbound tow. Tows with greater power for the load can develop more rudder control and would assume a smaller deflection angle and require less channel width. Also, tows with greater maneuverability because of independent operation of their screws, specially designed rudders, or auxiliary steering devices will require less channel width than would be indicated by the results of the model tests.

5.4.4 Irregular Bank Line

The preceding information on navigation in bends is based on bends having continuous concave banks. In natural streams having erodible beds and banks, the alignment of the concave banks might include scallops or landward indentations caused by erosion or local bank failures. These irregularities could have an adverse effect on tows navigating the bends and should be considered in the design of the projects. Model studies have indicated that small irregularities in the bank lines would have little or no effect on navigation but that longer irregularities in the downstream one-third of bends would generally have a significant adverse effect. These studies indicate that scallops in the bank line could be hazardous, particularly for downbound tows, when the length of a scallop along the bank is a minimum of 1 tow length and extends into the bank at least the width of the tow at a depth of 75% of the draft of the tow. This type of hazard is caused by currents moving into the scalloped area having a tendency to ground downbound tows moving close along the concave bank or causing them to hit the bank near the lower end of the scallop. In reaches where bank failure or erosion of this magnitude can be expected, the bank should be protected along a selected alignment. Existing scallops might require some remedial measures such as filling in the affected areas, adding dikes to prevent or reduce the movement of currents into the areas, or increasing the width of the channels to permit tows to move a sufficient distance from the affected bank lines.

5.4.5 Basis of Design

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

5.5 BRIDGE LOCATION AND CLEARANCES

5.5.1 Location

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

5.5.2 Clearances

The navigation span (horizontal clearance between piers) should be somewhat greater than the designed width of the channel in the reach depending on the alignment and velocity of currents in the reach, alignment of the channel approaching the bridge particularly from upstream, and the probable effects of the prevailing winds. The vertical clearance should be sufficient to permit tows to clear the low members of the bridge within the navigation span at the maximum navigable flow. Bridge clearances, both horizontally and vertically, are the responsibility of the US Coast Guard and planning should be coordinated with the local district of that organization.

5.5 SOURCE

The majority of this chapter was extracted from the US Army Corps of Engineers Manual, EM 1110-2-1611, *Layout and Design of Shallow-Draft Waterways*.

Chapter 6

OPEN-RIVER NAVIGATION

6.1 GENERAL

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

6.2 COST

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

6.3 FACTORS AFFECTING NAVIGATION

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

6.4 FEASIBILITY STUDY

The feasibility study should consider all of the factors that could affect navigation and cost. This study should include analyses of:

a. frequency and duration of river stages and discharges based on existing records,
b. channel width, depth, and alignment available, particularly during low flows,
c. composition of the bed and banks,
d. sediment characteristics of the stream and changes produced by variations in stages and discharge,
e. training and stabilization structures and corrective dredging required and their effects on sediment movement, currents affecting navigation, and the flood-carrying capacity of the stream,
f. type and volume of traffic that could be developed and justified with various improvement plans,
g. navigation periods that would be affected by unusually wet or dry years and ice, and
h. model studies required to determine problems that will be encountered and types and amount of training structures required, and to develop plans for the improvement of critical reaches.

6.5 SOURCE

The majority of this chapter was extracted from the US Army Corps of Engineers Manual EM 1110-2-1611, *Layout and Design of Shallow-Draft Waterways*. Additional information and examples can be found in this Manual.

Chapter 7
CHARACTERISTICS OF NATURAL STREAMS

7.1 GENERAL

7.1.1 Natural Streams

Natural streams can be characterized by their tendency to meander and migrate, irregularity and changing geometry, varying stage and discharge, and variations in the composition of beds and banks. Because of these variations, no two reaches are exactly the same. Many of the problems encountered in the development and improvement of natural streams are concerned with channel alignment and the movement of sediment into and within the stream. Scouring of the bed and banks and deposition in critical areas can affect channel depth and alignment and the operation and use of facilities and structures for navigation such as locks, harbors, docking areas, and other facilities such as hydroplants, sewage systems, and water intakes. Sediment movement can also affect the capacity of the channel to pass flood flows.

7.1.2 Sedimentation Problems

Since sedimentation problems can affect the type of waterway that could be developed and construction and maintenance cost, it is important that they be recognized and considered. The movement of sediment in natural streams is extremely complex depending on many factors, most of which are interrelated. Solution of sedimentation problems requires a knowledge of the general characteristics of the stream and of the principles of river sedimentation processes.

7.1.3 Sediment Load

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

7.1.4 Third Dimension

The movement of sediment in a stream has to be considered in three dimensions. The third dimension is provided by the Franco principle of lateral differential in water level. This principle is stated as follows:

> When conditions are such that a lateral differential in water level (or transverse slope) exists or is produced by changes, there will be a tendency for at least some of the total flow to move toward the lower elevation; the slower moving, sediment-laden bottom currents can make the change in direction easier than the faster moving surface currents and account for the greater concentration of sediment moving toward the lower elevation.

This general principle is involved in many of the developments in alluvial streams including the development of sandbars on the convex side of bends, movement of sediment around the end of and behind dikes, development of cutoffs and divided channels, shoaling in lock approaches, and so on. In each case there is either a buildup in water level on one side or a reduction caused by channel enlargement, contraction, or flow diversion that causes some of the flow to change direction.

7.2 SHOALING PROBLEMS

7.2.1 Deposition

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

7.2.2 Stage and Discharge

Changes in the discharge and stages produce changes in currents and in the movement of sediment that render the application or development of design principles extremely difficult. Model and field investigations have indicated how channel depths and configurations can be altered with change in stages. The movement of sediment in one reach can be considerably higher than in a reach just downstream during low flows and considerably lower during high flows. When one reach is not capable of moving the entire sediment load, shoaling will occur in that reach until velocities, slopes, and carrying capacity of the channel increase to that required to move the load.

7.2.3 Low-Water Profiles

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

7.2.4 Meandering Channels

Natural streams having erodible bed and banks will tend to meander, developing a sinuous course consisting of a series of alternate bends and crossings with some relatively straight reaches. The degree of sinuosity assumed by these streams depends on many factors including discharge, sediment load, valley slope, and composition of bed and banks. Unless the meandering of these streams is resisted by stabilization and training works, the bends will tend to migrate and change through the erosion and caving of their banks and the process of channel erosion and deposition. The channel is deeper in bends along concave banks and shallower in crossings and straight reaches (Figure 7-1).

7.2.5 Scour in Bends

The channel in bends tends to deepen during high river stages. Scour generally starts near the upper end of the bend and progresses toward the downstream as discharge and river stages increase. The increase in depth can be as much as one half to more than the amount of increase in stage, depending on the curvature of the bend and alignment of the channel upstream. With other conditions remaining the same, the increase in depth appears to be more a function of the river stage and stage duration than of the rate of change in stage. Where depths increase with river stage, shoaling of the channel starts during the falling stages near the upper end of the bend and continues toward the downstream during the low-water period.

7.2.6 Sediment Movement

The scouring of the channel in bends can cause a large amount of sediment to move into the crossing and reach just downstream. Because of the concentration of high-velocity currents and turbulence in bends, much more sediment can be moved in sinuous channels than can be moved in straight channels with the same average velocity and slope. For this reason straight channels and crossings downstream of a bend will tend to be shallow and unstable. Low-water slopes through bends are generally lower than the average because of the backwater effect produced by the shallow crossing downstream (Figure 7-1d). Because of the reduced slopes and velocity, deposition occurs in bends during low flows. However, the amount of deposition is seldom sufficient to reduce depths to less than that required for navigation.

7.2.7 Crossings

In meandering streams the low-water channel in the straight reach between alternate bends crosses from one side of the river to the opposite side. Because the movement of sediment in a bend is greater than the capacity of the straight channel downstream during the higher flows, deposition occurs in the crossing, limiting depths available for navigation. As river stages decrease, slopes and velocities over the crossing tend to increase, increasing the movement of sediment and depths. The rate of scour and depths available for navigation depend on the stage and stage duration. After a prolonged high-water period or after a rapid decrease in stage, depths over crossings will tend to limit channel depths available and are a

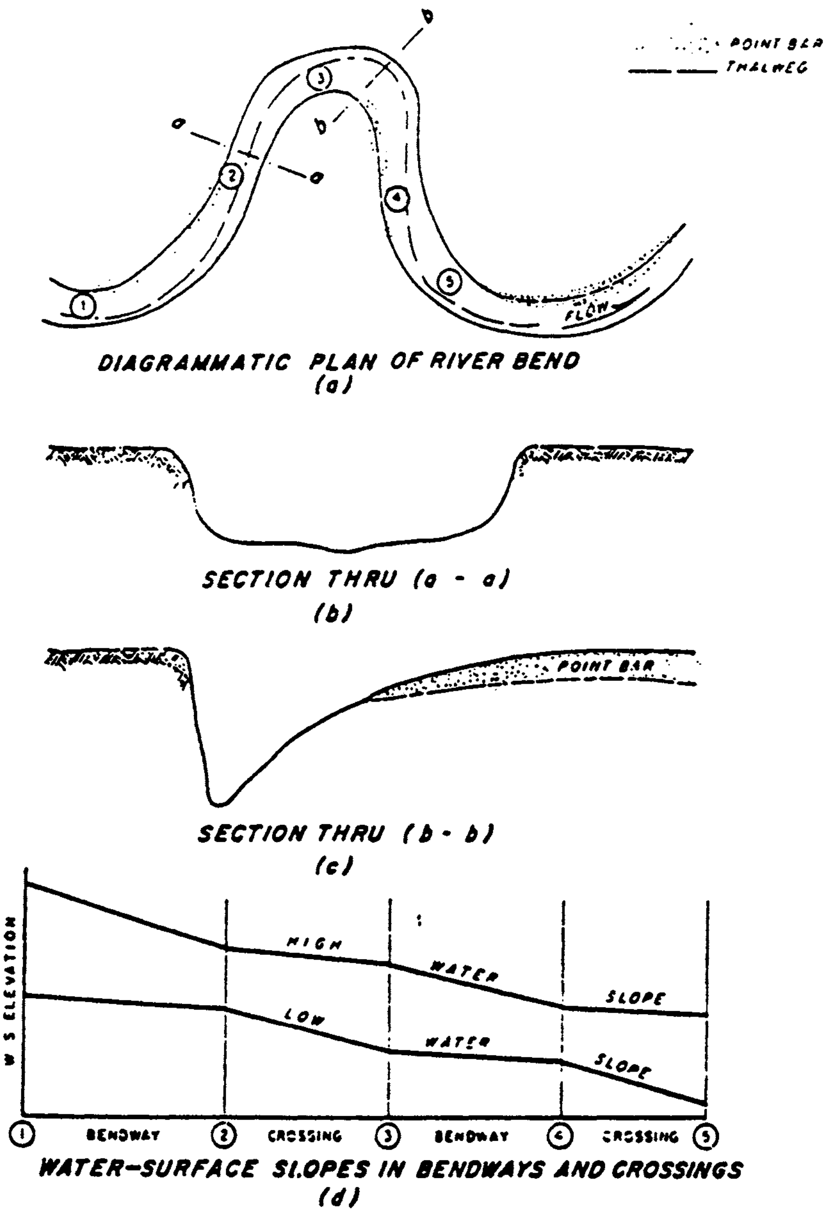


FIGURE 7-1. *Characteristics of a River Reach.*

frequent source of navigation difficulties. Alignment and depth of the channel in crossings depend on variations in flow conditions and alignments of the reaches upstream and downstream. Maintaining a satisfactory channel in crossings will be more troublesome if regulating structures on the concave side of the bend upstream are not carried far enough downstream to prevent dispersion of the higher flows and if the crossings to the next bend are relatively long. Extending the training works in a bend toward the crossing downstream improves the alignment and depth of the channel over the crossing and flow into the bend downstream.

7.2.8 Straight Channels

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

7.2.9 Divided Channels

Bifurcated channels or divided flow will be found in many alluvial streams in addition to those formed by cutoffs. Side channels will tend to carry a greater proportion of the sediment load than the proportional discharge, because of the lateral differential in water level which depends upon the shape, size, and angle of entrance with respect to the direction of flow from upstream and the relative lengths of the two channels. When the entrance to the side channel is wide in comparison with the rest of the channel, sediment will tend to be deposited near the entrance, which could eventually reduce or eliminate flow through the channel during low stages. Depths in the main channel will tend to be limited when side channels carry a sizable proportion of the total flow, and the partial or full closure of these channels will be required to improve depths in the main channel. When deposition occurs near the entrance, the sediment-free flow moving downstream of the entrance could cause scouring and deepening of the side channel and bank caving. When there is a substantial amount of flow diverted through a side channel, the main low-water channel will tend to develop toward the point of diversion (Figure 7-2).

7.2.10 Tributary Streams

Flow from tributary streams causes a local increase in water level just upstream and channelward of the inflow and a lowering of the water level

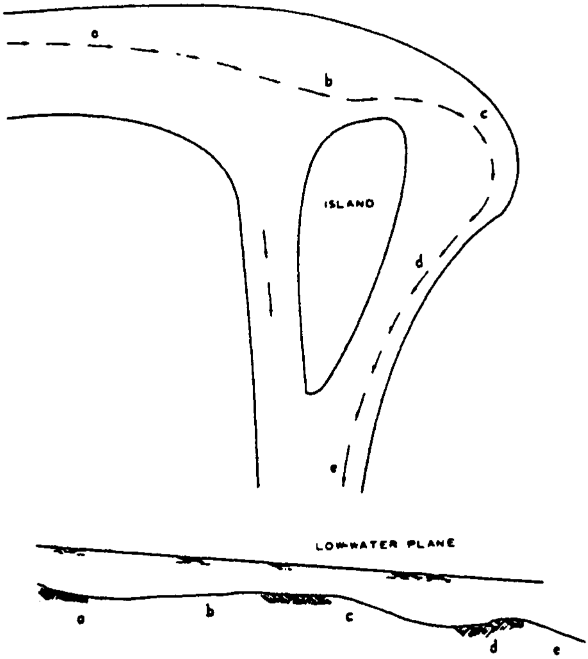


FIGURE 7-2. Typical Divided Channel.

along the adjacent bank downstream. The difference in water level will depend on the discharge, and current direction and velocities of the flow entering the main stream. Because of the lateral differential in water level created, there will be a tendency for shoaling along the adjacent bank downstream and for sediment carried by the tributary to be moved along that side of the channel. Accordingly, the deeper channel will tend to form away from the adjacent bank (Figure 7-3).

7.2.11 Entrances to Canals and Harbors

Entrances to canals and slackwater harbors involve openings in the bank line and a local increase in the channel width. This causes a lowering of the water level at the entrance and a tendency for bottom currents and sediment to move toward the entrance, resulting in a tendency for shoaling. The amount of shoaling will depend on the amount of sediment carried by the stream, size of the entrance, and location of the entrance with respect to the alignment of the stream channel. Shoaling in the entrance could also be affected by the rate of rise and fall of river stages that cause flow toward and away from the canal or harbor (Figure 7-4).

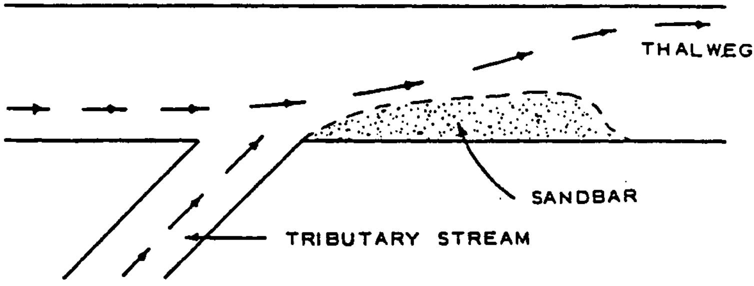


FIGURE 7-3. Effect of Tributary on Channel Configuration.

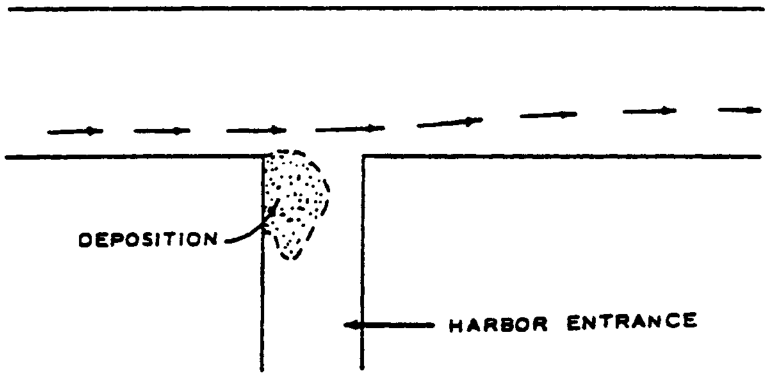


FIGURE 7-4. Shoaling in Harbor Entrance Located Along Riverbank.

7.3 SOURCE

The majority of this chapter was extracted from the US Army Corps of Engineers Manual EM 1110-2-1611, *Layout and Design of Shallow-Draft Inland Waterways*.

Chapter 8

RIVER TRAINING WORKS

8.1 GENERAL

8.1.1 Requirement

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

8.2 DREDGING

8.2.1 Corrective Dredging

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

8.3 CHANNEL STABILIZATION

8.3.1 Bank Erosion

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

8.3.2 Types of Protection

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

8.4 CUTOFFS

8.4.1 Purpose and Method

Cutoffs are used to eliminate sharp bends, eliminate troublesome reaches, reduce the length of the navigation channel, or increase the flood-carrying capacity of the stream. Cutoffs are usually formed by dredging a pilot channel across the neck of one or more bends. The size, slope, and alignment of the pilot cut should be such that the cutoff will develop naturally to take most or all of the flow of the stream. The rate of development of a cutoff depends on the erodibility of the material through which the cutoff is made, size and shape of the pilot cut, length of the cutoff with respect to length of the channel around the bend, and location of the entrance with respect to the alignment of the existing channel. The rate of development of a cutoff can be increased by the gradual closure of the old bendway channel or by structures designed to increase the tendency for shoaling in the upper end of the existing bend and to direct flow toward the pilot cut.

8.4.2 Old Bendways

In planning cutoffs, the use of the old bendway for recreation and/or harbor facilities should be considered. In many cases, the general practice has been to close off the upper end of the old bend with a closure dike or embankment to eliminate the movement and deposition of sediment in the bend. Structures will usually be required in the lower end of the old bend to reduce the tendency for shoaling and the need for maintenance dredging (Figure 8-1).

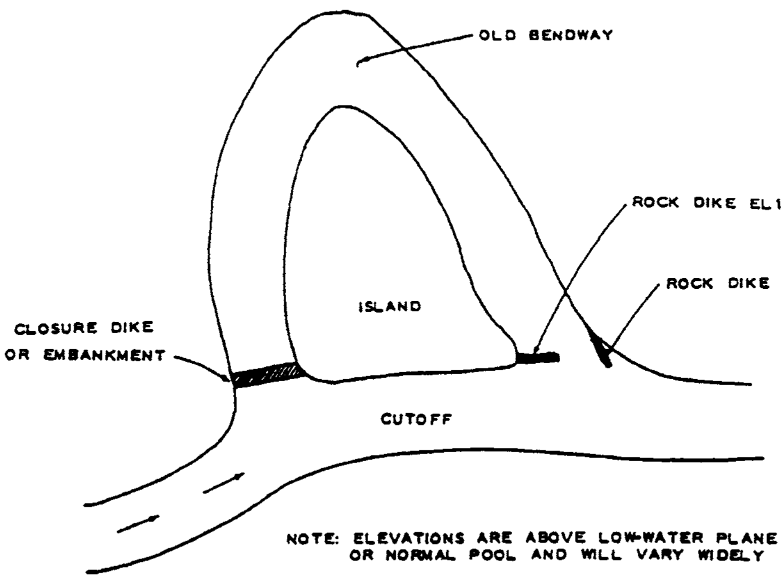


FIGURE 8-1. Cutoff—Old Bendway Used for Harbor or Recreation.

8.5 TRAINING STRUCTURES

8.5.1 General

The development and improvement of channel alignment and depth often require the use of structures to reduce the width of the channel, realign the channel, and stabilize the low-water channel. These structures usually consist of some type of dikes constructed of timber pile clusters, stone, or piling with stone fill. The type or types of structures used and their arrangements should be based on the characteristics of the stream, problem or problems to be resolved, and conditions contributing to the problem. The design of the structures should consider the effects of the structures on currents existing in the reach and the movement of sediment, and the effects of the resulting currents on navigation.

8.5.2 Spur Dikes

The most common type of structure used in channel improvement and development extends from the riverbank channelward in a direction approximately normal to the channel being developed (Figure 8-2). These dikes are usually included in a system of two or more and are generally referred to as spur dikes. These dikes have also been referred to as transverse dikes, cross dikes, wing dams, jetties, and so on. Spur dikes should be designed to provide a favorable lateral differential in water level to increase their effectiveness and reduce construction and maintenance cost. Improperly designed spur dikes could be either ineffective or unstable, increase channel losses, cause shoaling upstream, or develop a channel of poor alignment.

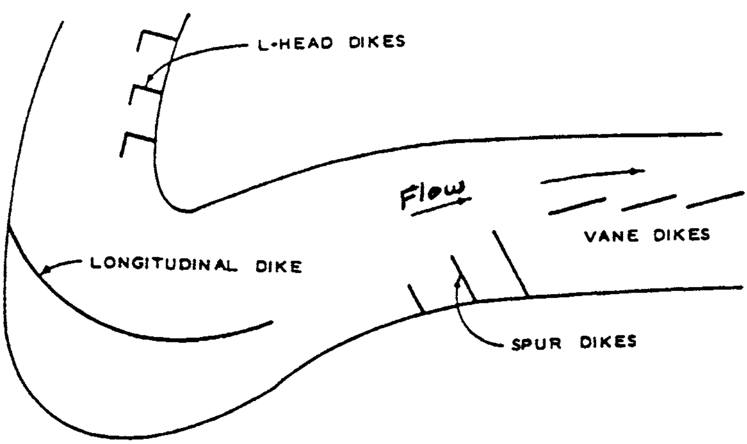


FIGURE 8-2. Types of Dikes in General Use.

8.5.3 Longitudinal Dikes

Longitudinal dikes are continuous structures extending from the bank toward the downstream generally parallel to the alignment of the channel being developed (Figure 8-2). Properly designed longitudinal dikes are the most effective type of structure in developing a stable channel but are also the most expensive. These structures can be used to reduce the curvature of sharp bends and to provide transitions with little resistance or disturbance to flow. However, once in place, it is difficult and expensive to change the alignment of the dike. If not constructed high enough, flow over the top of the dike would tend to move tows toward and against the structure. Little or no natural deposition can be expected landward of the dikes but the area could be used for the placement of dredged material. Consideration for fish and wildlife might indicate the need for modification, such as providing openings to develop and maintain water areas behind the dike.

8.5.4 Vane Dikes

Dikes placed in the form of a series of vanes have proved effective as a means of controlling channel development and sediment movement under certain conditions. These dikes consist of lengths of dikes located out from the bank with space in between and placed at a slight angle to the alignment of the currents to develop the lateral differential in water level desired (Figure 8-2). The length of the gaps between the dikes is usually about 50 to 60% of the length of each vane and should be placed where there is or will be movement of sediment. These dikes, developed as a result of model studies, have been used successfully on the Mississippi and Arkansas Rivers. Vane dikes can be used independently or as extensions to spur dike systems. Vane dikes are much cheaper than conventional dikes since they can be placed in relatively shallow water generally parallel to the channel limit line and produce little disturbance to flow. Vane dikes placed just upstream of spur dikes or landward of the channel ends of spur dikes would not develop the lateral differential in water level desired and should be avoided.

8.5.5 L-Head Dikes

L-head dikes are spur dikes with a section extending downstream from the channel ends generally parallel to the channel line (Figure 8-2). The L-head section can be used when the spacing between dikes is too great, to reduce scour on the end of the spur dike, or to extend the spur dike system farther downstream. L-heads tend to block the movement of sediment behind the dike and when the crest is lower than the main dike, permit surface currents over its top and cause scour on the landward side. These dikes could be designed to reduce shoaling in harbor entrances or to maintain an opening in the lower end of a bypassed channel. Notches in the land connection section have been successful in providing enhanced fish habitat by allowing controlled flow behind the dike.

8.5.6 Closure Dikes

In reaches where there are islands and divided flow, depths will tend to be limited. In such cases, it will be necessary to reduce or eliminate the low and medium flows from all but the principal channel being developed. This is accomplished by diverting sediment into the side channels or by closure structures across the side channels. Sediment can be diverted into the side channel by developing the lateral differential in water level with spur dikes, vane dikes, or by a combination of both. When the length of the side channel is short relative to that of the main channel as in a bend, closure dikes across the shorter channel will tend to be difficult to maintain because of the high head that develops across the dike, producing scour downstream. In such cases, one or more closure dikes placed downstream at successively lower elevations will tend to divide the total drop between the dikes and reduce the amount of scour that would tend to endanger a single structure (Figure 8-3).

8.5.7 Bendway Weirs

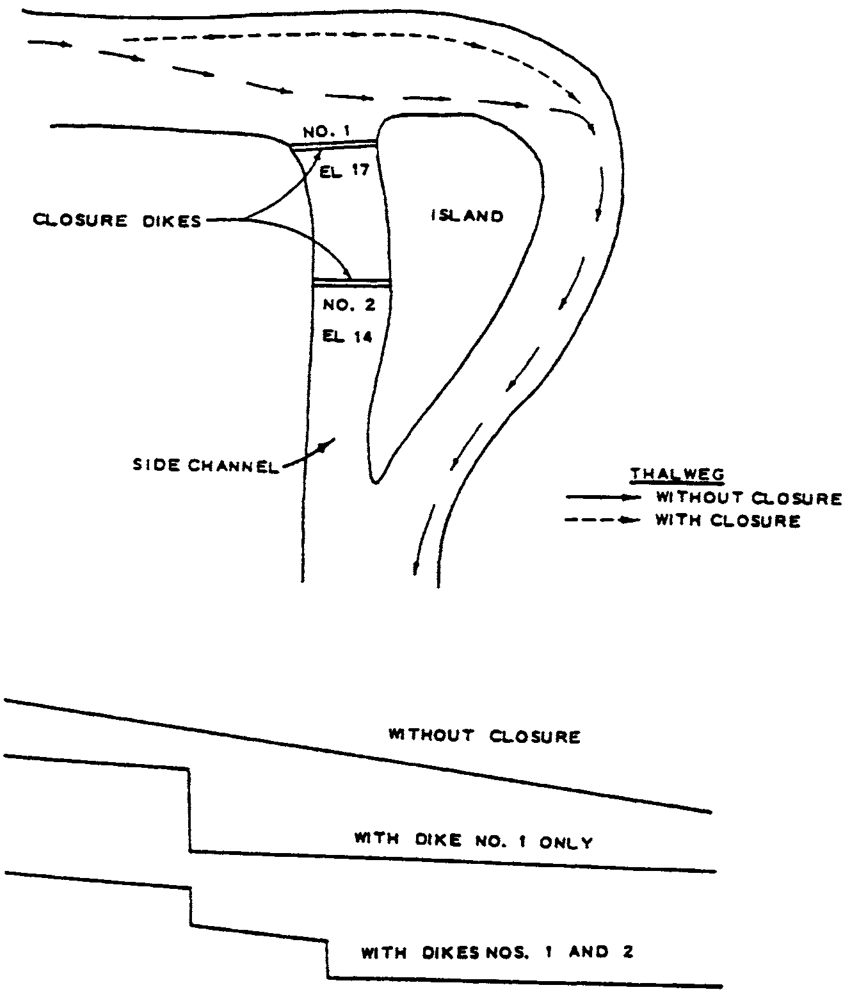
Totally submerged stone weirs along the outside of a river bend are a new concept of river training developed by the US Army Corps of Engineers. Figure 8-4 shows this bendway weir concept. The river currents are directed more toward midchannel instead of digging the channel deeper on the outside of the bend. Sediment is now captured between the weirs rather than building a sand bar on the inside of the curve. The weirs are submerged with sufficient water over them to allow barge passage even during low water conditions. The weirs have an upstream orientation of a 30 degree angle to the main river flow. In 1990 the Army Corps of Engineers installed a series of 13 weirs on the Mississippi River along Dogtooth Bend, about 160 miles south of St. Louis. This weir installation has improved navigation with elimination of costly traffic delays. Instead of waiting for tows to slowly navigate the treacherous bendway one by one, traffic can steer straight through the bend without delays. Another benefit is reduced maintenance dredging.

8.5.8 Examples of River Training Works

The traditional method of constricting a channel for navigation is construction of dikes and revetments. Figure 8-5 shows an idealized plan. Figures 8-6 through 8-11 show development of a controlled navigation channel at Rock Bluff Bend on the Missouri River from 1934 to 1983. Other examples of river training works are shown on Figures 8-12 through 8-14.

8.6 SOURCE

The majority of this chapter was extracted from the US Army Corps of Engineers Manual EM 1110-2-1611, *Layout and Design of Inland Waterways.*



WATER-SURFACE PROFILES (SIDE CHANNEL)

FIGURE 8-3. Side Channel Closure.

FIGURE 8-4. Bendway Weir.

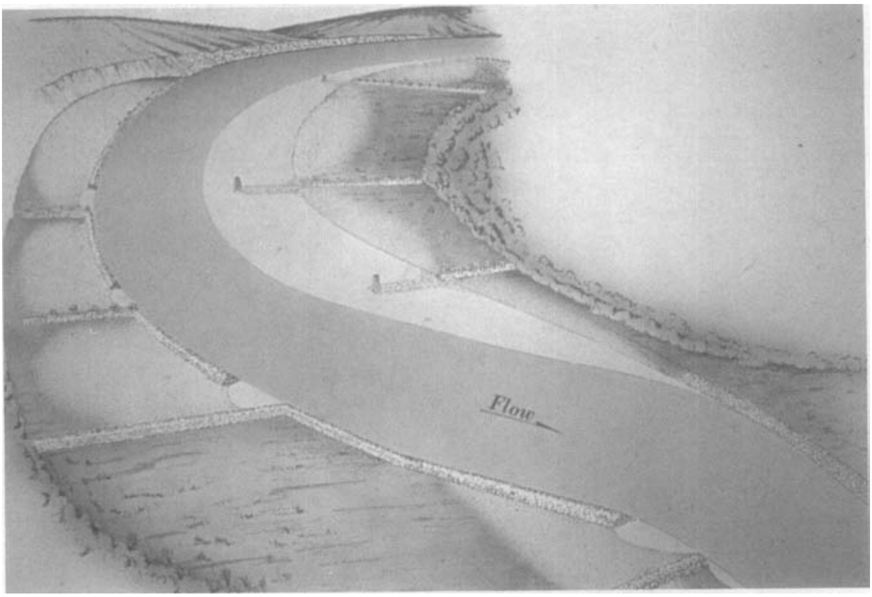


FIGURE 8-5. Idealized River Training Plan.

FIGURE 8-6. Uncontrolled Section of Missouri River at Rock Bluff Bend, September, 1934.

FIGURE 8-7. Timber Pile Dike Field, March, 1935.

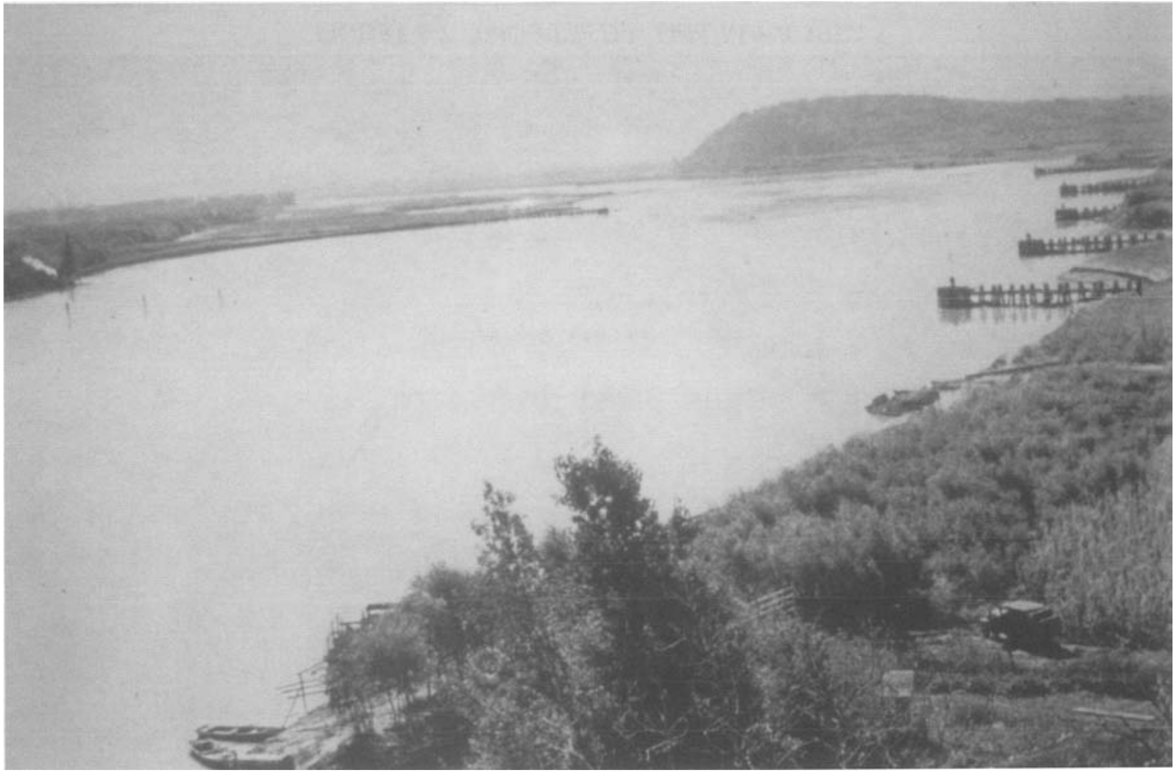


FIGURE 8-8. Dike Field Partly Filled, October, 1939.

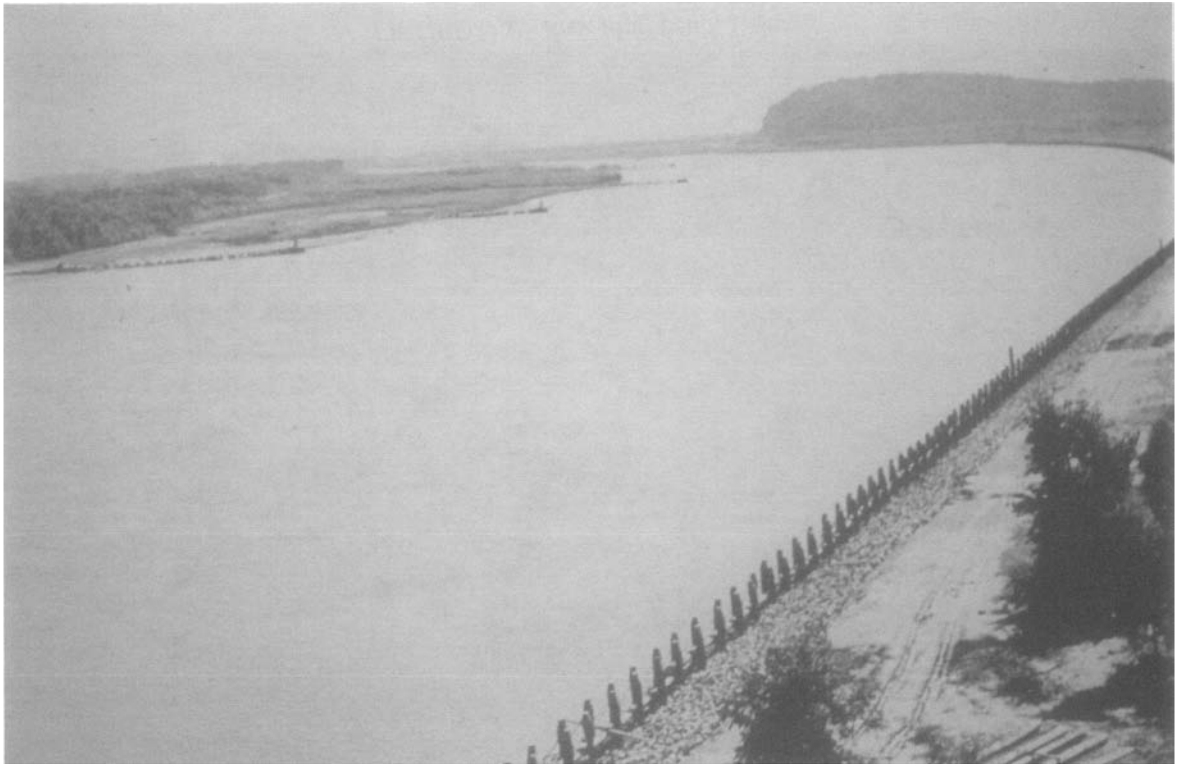


FIGURE 8-9. Revetment Along Filled Dike Field, Dike Field on Opposite Shore, September, 1942.

FIGURE 8-10. Rock Bluff Bend, 1956.

FIGURE 8-11. Rock Bluff Bend, March, 1983.

FIGURE 8-12. Box Cut Pilot Channel on Red River, 1988.

FIGURE 8-13. Channel Realignment on Red River, 1989.

FIGURE 8-14. Stone Dike Field on Missouri River, 1987.

Chapter 9

LOCK AND DAM SYSTEMS

9.1 FACTORS INVOLVED

9.1.1 General

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

9.1.2 Locks in Stream Channels

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

9.1.3 Other Considerations

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

9.2 CHANNEL ALIGNMENT

9.2.1 Effects of Channel Alignment

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

9.2.2 Locks on Concave Side of Bend

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

9.2.3 Locks on Convex Side of Bends

Locks located on the convex side or inside of a bend would affect less of the total river flow but would require downbound tows to make a turn for the approach that would place the stern riverward of the bank line in currents moving toward the spillway. Also, there would be a tendency for shoaling on the convex side of the channel. Many accidents and delays have been experienced by downbound tows attempting to approach locks on the convex side of the bend. Therefore this site should be avoided. An example of the problems with a lock on the inside of the bend is the old Gallipolis Lock on the Ohio River. A unique series of photos captured an accident in progress at the Gallopolis Lock. Figure 9-1 shows a tow partly in the lock approach but being pulled toward the spillway. Figure 9-2 shows the tow wrapped around the lock wall. Figure 9-3 shows part of the tow drifting toward the spillway and the tug trying to round up the free floating barges. The old Galliopolis Lock has since been replaced by the Robert C. Byrd Lock which is shown in Figure 9-10.

9.2.4 Bypass Canals

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

FIGURE 9-1. Tow Attempting to Enter Old Gallipolis Lock on Ohio River.

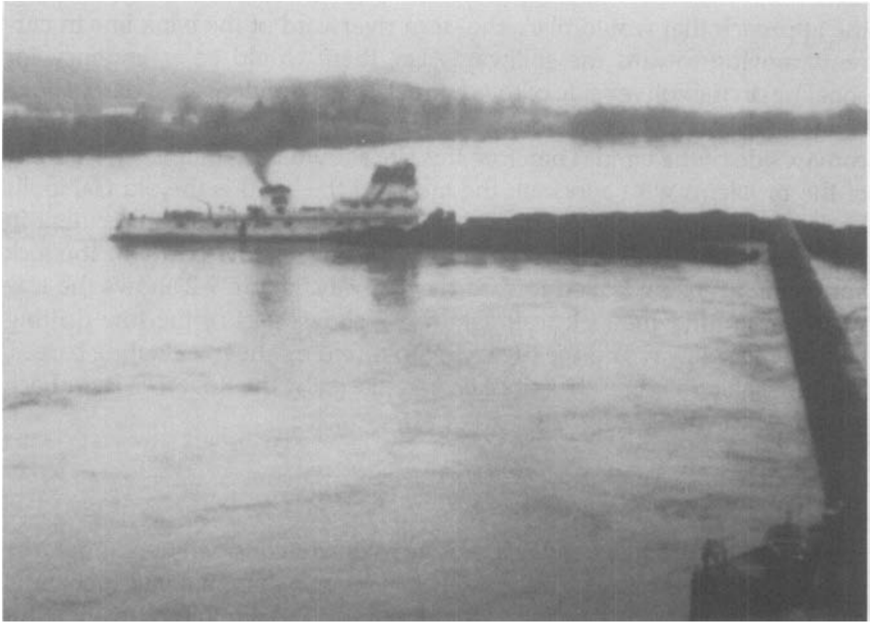


FIGURE 9-2. Tow Being Pulled Around Guidewall Toward Spillway.

FIGURE 9-3. Tugboat Retrieving Barges.

9.2.5 Factors to be Considered

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

9.3 LOCKS IN CANALS

9.3.0 Effects on Navigation

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

9.3.2 Upper Canal Entrance

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

9.3.3 Two-Way Traffic

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

9.3.4 Flow Across Adjacent Overbank

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

9.3.5 Lock Filling

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

9.3.6 Reduction of Surges in Canal

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

9.3.7 Upper Lock Approach

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

9.3.8 Lower Lock Approach

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

9.4 LOCK AUXILIARY WALLS

9.4.1 Guide Walls

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

9.4.2 Upper Guide Wall

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

9.4.3 Lower Guide Wall

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

9.4.4 Guard Walls

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

9.4.5 Upper Guard Wall

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

9.4.6 Lower Guard Wall

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

9.5 ARRANGEMENT OF LOCKS AND AUXILIARY WALLS

9.5.1 Single Lock

Walls used to assist tows in approaching and entering the locks vary in type and arrangement. In their simplest form, single locks might include guide walls, guard walls, or a combination of both. Guide walls are usually on the land side of the lock (Figure 9-4(a)); guard walls are usually on the river side (Figure 9-4(b)). Some locks have an upper guard wall and a lower guide wall (Figure 9-4(c)). The upper gate pintles of most locks are along the axis of the dam so as to place the lock chamber in the lower pool, reducing pressure on the lock walls when the lock is dewatered. When the guard wall

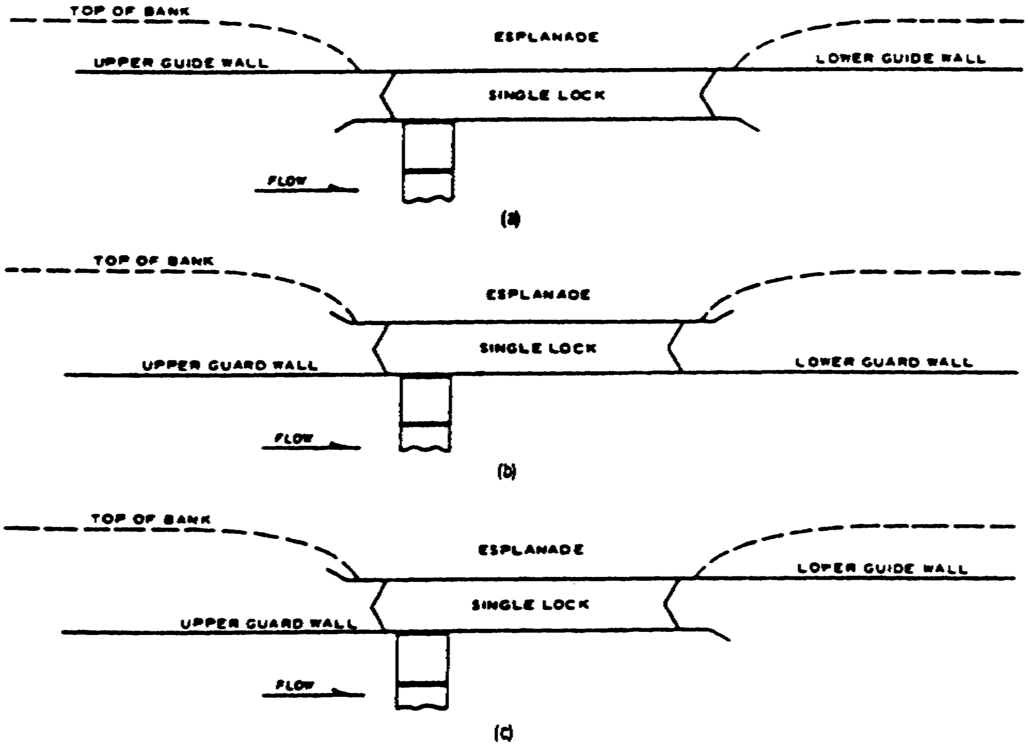


FIGURE 9-4. Arrangement for a Single Lock.

is of sufficient length, it also serves as a guide wall for the lock. The preferred upper guard wall configuration is shown in Figure 9-4(b) and (c). The wall location prevents the tow from being drawn toward the spillway and causing a misalignment with the lock. Figure 9-5 shows the preferred guard wall configuration at the new Bonneville lock which was completed in 1993.

9.5.2 Adjacent Locks

With two adjacent locks, there is a common intermediate wall. The general practice has been to equip the riverside (main) lock with a ported upper guard wall and a solid lower guard wall. The landside (auxiliary) lock usually has an upper guide wall (land side of the lock) and a lower guide wall. When the landside lock is shorter than the riverside lock, the landside face of the intermediate wall extending beyond the end of the auxiliary lock could be used as the guide wall (Figure 9-6(a)). When the upper guard wall is ported, tows tend to be moved toward the guard wall because of flow through the ports, making it somewhat difficult for downbound tows to approach the guide wall for passage through the landside lock. The diffi-

FIGURE 9-5. New Bonneville Lock, Columbia River.

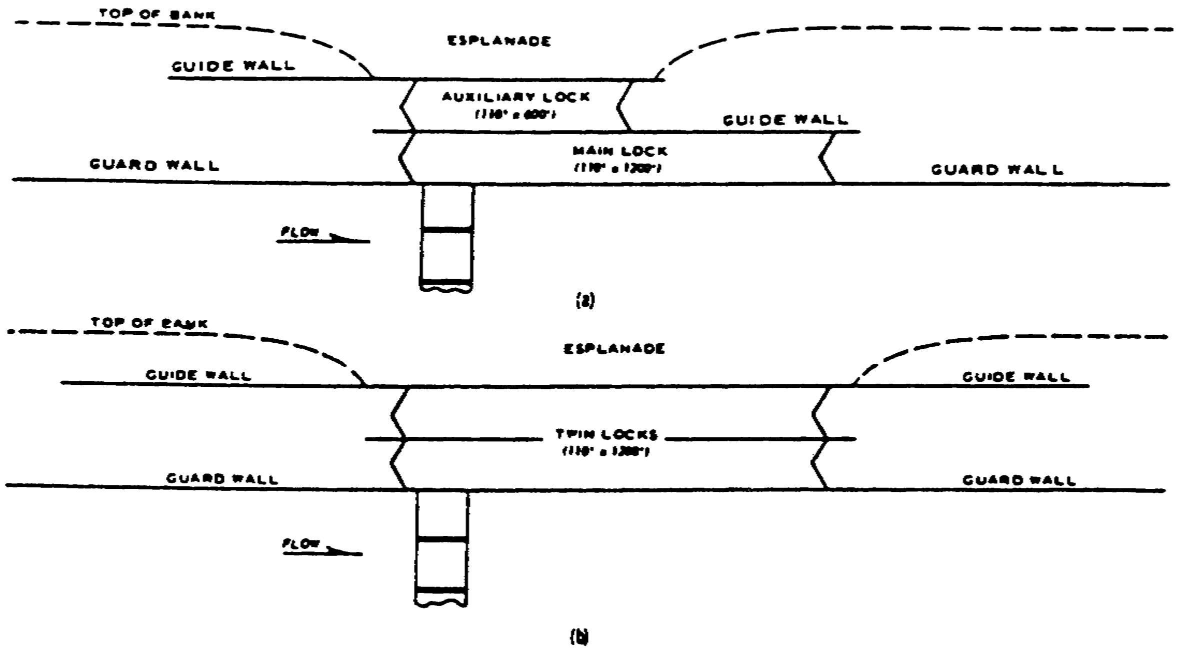


FIGURE 9-6. Arrangement with Adjacent Locks.

culty is increased when the landside lock is the same size as the riverside lock. Another problem confronted with this arrangement is that a tow cannot safely approach either lock when another tow is leaving or tied up along the guide or guard wall, resulting in delay for approaching or departing tows (Figure 9-6(b)).

9.6 NEW ARRANGEMENTS OF LOCKS AND AUXILIARY WALLS

9.6.1 General

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

9.6.2 Upper Lock Walls with Adjacent Locks

The new concept is to provide an upper guard wall for both locks when the locks are adjacent (Figure 9-7(a)). Both guard walls would have to be ported. The landside guard wall should be at least half the length of the usable portion of the lock chamber and the riverside guard wall should be of sufficient length to extend at least three fourths of the length of the usable portion of the lock chamber beyond the end of the guard wall for the landside lock. These lengths are based on limited tests with specific projects and some variations might be desirable, depending on local conditions. The same arrangement could be used with adjacent locks of different sizes with the upper gate pintles of both locks along the axis of the dam. Since there would generally be little flow through the ports in the landside lock guard wall, the tops of the ports should be a few feet higher than those in the riverside wall to develop currents that would assist tows in approaching the wall. As a result of this arrangement, a downbound tow could approach the riverside lock and be followed by a downbound tow approaching the landside lock as soon as the tow using the riverside lock landed along the guard wall. Also, a downbound tow using the landside lock can approach the lock while an upbound tow is leaving the riverside lock; and a downbound tow could approach the riverside lock while an upbound tow is leaving the landside lock, provided the head of the upbound tow does not extend beyond the end of the landside guard wall. An example of the Figure 9-7(a) upstream wall configuration is Smithland lock shown on Figure 9-8. The Melvin Price Locks on the Mississippi River near St. Louis are an example of the Figure 9-7(b) separated lock configuration.

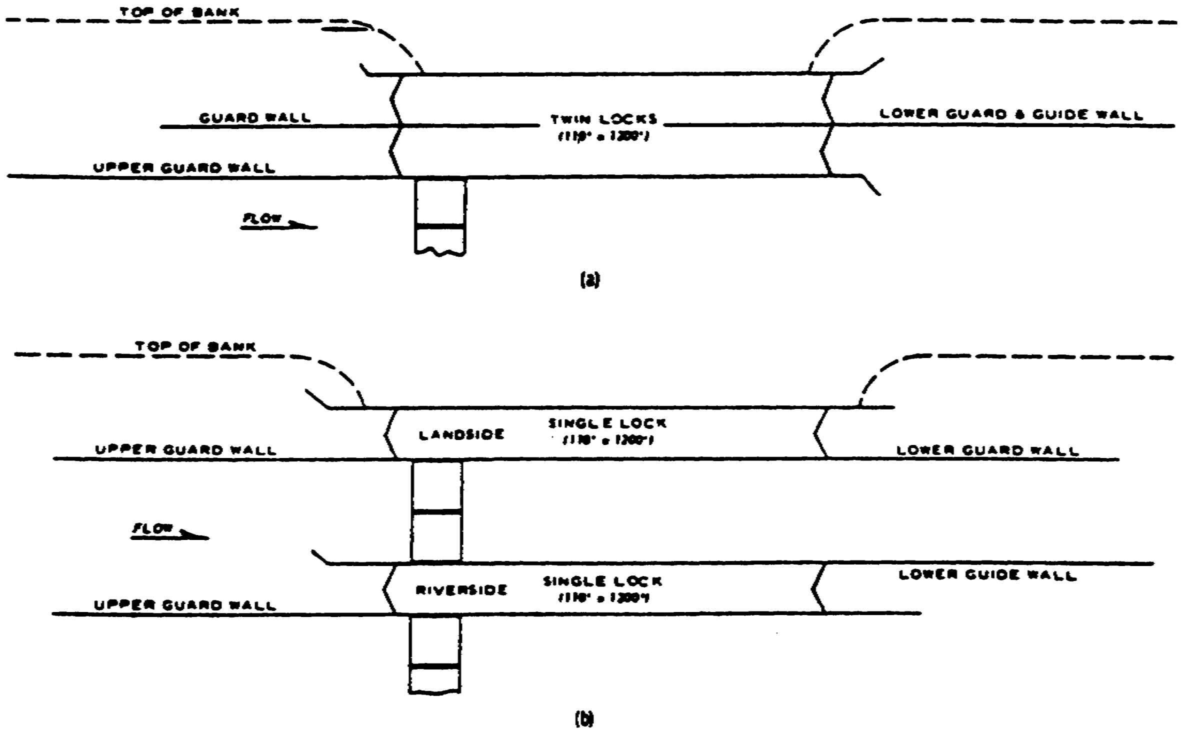


FIGURE 9-7. New Concepts in Lock Arrangement.

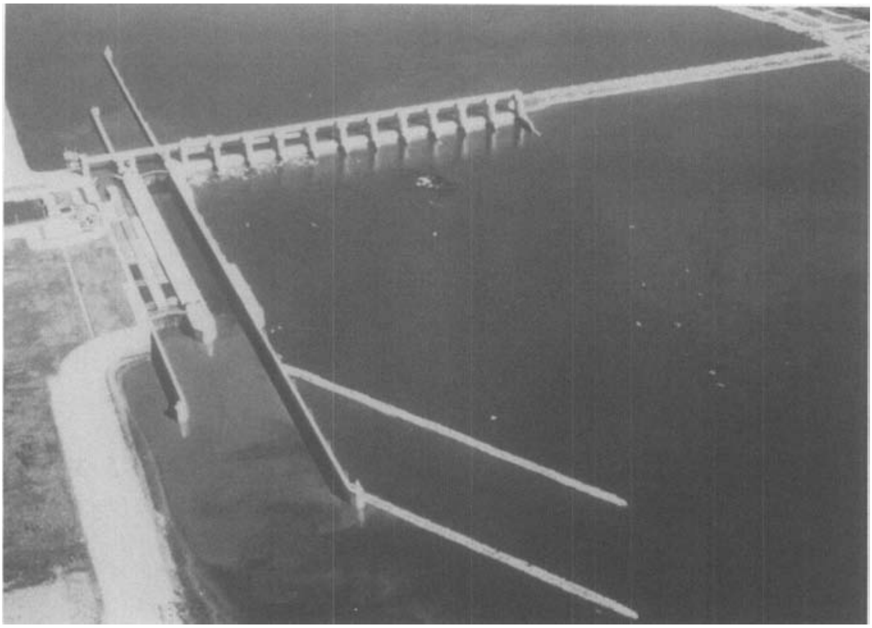


FIGURE 9-8. Smithland Locks, Ohio River.

9.6.3 Lower Lock Walls with Adjacent Locks

In the lower approach, the new arrangement provides for the extension of the intermediate wall to form a guide wall for both locks (Figure 9-7(a)). The landside and riverside faces of the wall would have to be in line with the inside faces of the adjacent locks and constructed to withstand the impact of tows approaching the wall from either side. With this arrangement, tows entering or leaving one lock would not interfere with tows entering or leaving the other lock. For safe two-way traffic, the length of this wall should be the same as that of the tows using the locks. In addition to the advantage of two-way traffic, a long intermediate wall would cause a more gradual increase in channel width than with a long riverside guard wall, thereby reducing the shoaling tendency and, in turn, maintenance cost and interference with traffic during maintenance dredging. Shoaling, if any, would start in the approach to the riverside lock where it could be removed without interfering with traffic using the landside lock. The only disadvantage attributed to this scheme is that upbound tows approaching the riverside lock would use more power since they would be moving farther out into the channel in higher velocity currents and would encounter some currents along the river side of the wall. However, this disadvantage is more than offset by the elimination of delays and power required to maintain position in the stream while waiting for other tows to clear the locks.

9.6.4 Separation of Locks

The second concept involves separation of the locks to provide two-way traffic in either or both directions (Figure 9-7(b)). The amount of separation required is presently a matter of opinion and could vary depending on local conditions. Navigation interests indicated that a separation of about 270 ft was acceptable for the Melvin Price Locks, Mississippi River. Based on the movement and passing of tows through restricted reaches and bridge spans and on results of model studies, it appears that separations of about 200 ft or less might be adequate under most conditions. Separation of the locks would produce a greater obstruction to flow and result in an increase in crosscurrents in the lock approaches. To reduce the effects of the obstruction, spillway gates should be provided between the locks to pass some of the flow affected by the locks. This would reduce the crosscurrents produced by the total flow moving toward the spillway across the riverward lock approach; the size and intensity of the eddy that would develop in the lower approach would also be reduced since the amount of channel expansion would be reduced. The hydraulics involved in the development of satisfactory navigation conditions with lock separation are more complex than for adjacent locks; design should not be finalized without benefit of a model study. Figure 9-9 shows the lock separation configuration for the new Melvin Price Locks.

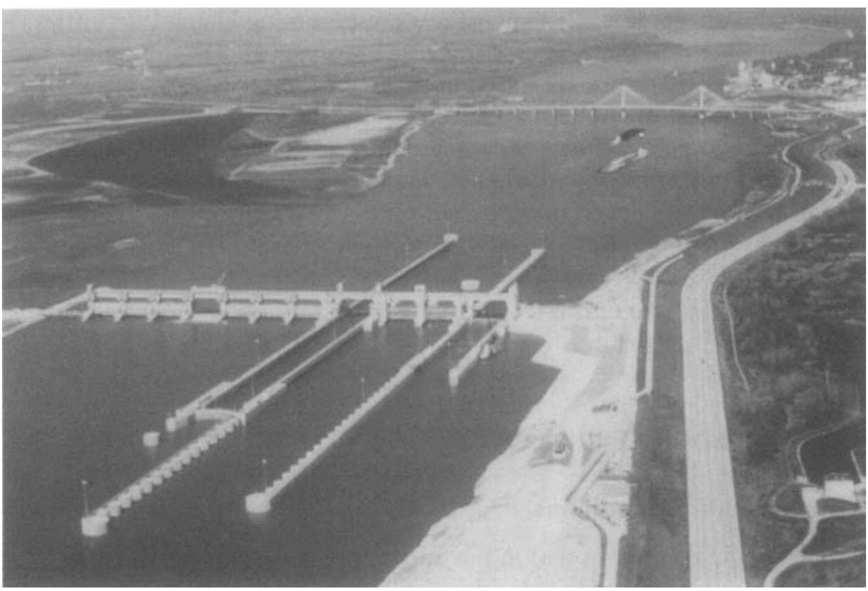


FIGURE 9-9. Melvin Price Locks, Mississippi River.

9.6.5 Upper Approach

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

9.6.6 Lower Approach

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

9.6.7 Locks in Canal

Locks located in a canal bypassing the dam in the main channel should be provided with guide walls to assist tows in becoming aligned for entrance into the lock. Since there are usually little or no currents in the canal, walls can be shorter than those required in the main channel, particularly with a single lock. When twin locks are located in the canal, extension of the intermediate lock wall can be used as guide walls in the upper and lower approach. This arrangement should be less costly than separate guide walls for each lock and could permit two-way traffic under most conditions because of the separation provided by the center wall. The Robert C. Byrd Lock on the Ohio River is an example of two locks in a bypass canal. The Figure 9-10 aerial photo of this lock is in the foreground and the old Gallipolis Lock is in the background.

9.7 UPPER LOCK APPROACH

9.7.1 Navigation Conditions

Navigation conditions within the upper approach to a lock can affect the safety of tows approaching or leaving the lock and the time required for tows to transit the lock. Conditions for downbound tows can be particularly hazardous if high current velocities and crosscurrents prevail. Properly designed

FIGURE 9-10. Robert C. Byrd Locks, Ohio River (Replacement for the Old Gallipolis Locks).

guard and guide walls can assist tows in approaching and leaving the lock. Locks with long guide walls and short guard walls will tend to have currents along the guide wall and crosscurrents near the end of the guard wall. Locks with long guard walls and short guide walls will tend to develop crosscurrents near the end of the guard wall. The intensity or the effects of crosscurrents can be reduced or eliminated by providing ports in the upper guard wall, by reducing velocities in the approach channel by means of structures, or by extension of the guard wall using cells. Conditions in the upper approach can also be affected adversely by the alignment of currents upstream of the guard wall and uneven channel depths in the lock approach.

9.7.2 Ports in Guard Wall

Ports in the upper guard wall eliminate or reduce crosscurrents near the end of the wall by permitting all or a major portion of the flow intercepted by the wall to pass under the wall. The effectiveness of the ports will depend on their number, size, and hydraulic efficiency based on the type of construction. As a general rule, the total cross-sectional area of the port openings should be equivalent to the total cross-sectional area of the approach channel affected by the locks and lock walls. Flow through the ports will tend to move a tow toward the wall, and could make it difficult for tows to pull away from the wall. This effect can be reduced by placing the top of each port below what would be the bottom of a loaded barge locking at normal pool, with a relatively deep channel between the guard wall and adjacent bank. In most cases, placing the tops of the ports four to six feet below the bottom of a loaded barge at normal pool would be adequate; at structures with hinged pool operation, the tops of ports might have to be somewhat lower. The bottom of the channel between the guard wall and the adjacent bank should be near or lower than the elevation of the bottom of the ports to reduce velocities for tows approaching the wall and prevent a buildup of head against the land side of the tow. In riverbeds of easily erodible material, the bottom of the ports should be protected from scouring, particularly those near the lower end of the wall. Without protection the ports near the lock would tend to scour and pass more of the flow with a corresponding reduction in flow through the ports near the upper end. Increasing the concentration of flow through the lower ports would increase the tendency for tows to be moved toward the wall; this could also affect flow through the dam gates near the lock.

9.7.3 Effects of Ports on Movement of Ice and Debris

With properly designed ports and approach conditions, tows can be made to drift into the lock approach and become aligned along the guard wall with little or no rudder control. However, ports in the upper guard wall will increase the tendency for ice and floating debris to be trapped in the lock approach. A long guide wall and short guard wall will reduce the amount of debris trapped in the lock approach but, at the same time, will generally preclude the use of an adequate number of ports to eliminate or substantially reduce crosscurrents near the end of the wall. When adjacent locks are used with a guide wall on the landside lock, currents moving toward the ported guard wall of the riverside lock could cause difficulties for downbound tows approaching the guide wall on the landside lock.

9.7.4 Channel Depths

Differences in depth in the approach channel can affect the movement of tows in the approach, particularly if the tow is moving at reduced speed from deep to shallow water. Tows moving along a bank and passing from a deep to a shallow portion of the channel block a portion of the flow in the shallow channel, causing a higher water level to develop between the tow and the adjacent bank that could move first the head and then the remainder of the tow riverward. The effects of changes in depths can be minimized or eliminated with submerged dikes or groins located some distance upstream of the lock walls. Submerged groins (dikes) can also be used to reduce velocities in the approach. The elevation and spacing of the groins would depend on channel depths and current direction and velocities. In previous studies, groins with crests 20 ft below normal upper pool elevation spaced 1 to 1½ times the length of the upstream groin have proved satisfactory (Figure 9-11). The groins should extend from the bank at least to a line forming an extension of the landside face of the upper guard wall. Dikes that are too high above the bed or spaced too far apart will tend to produce turbulence with erratic currents extending to the surface. These currents are usually local and have little or no effect on the movement of tows approaching or leaving the lock. The disturbance can be reduced by closer spacing of the dikes or by filling between dikes. A fill of the same elevation as the dikes would not be as effective in reducing velocities because of the reduction in channel roughness.

9.7.5 Overbank Flow

Overbank flow moving toward the river from the adjacent bank or from the river toward the adjacent bank can produce serious crosscurrents (Figure 9-11). This condition can occur with a low overbank and an embankment blocking downstream flow or with a low overflow embankment with high ground upstream causing some flow toward the overbank. This condition can be eliminated or reduced by constructing a fill or dike along the adjacent bank extending from the dam or locks far enough upstream where tows can either avoid the currents created by the flow or maintain speed and rudder power

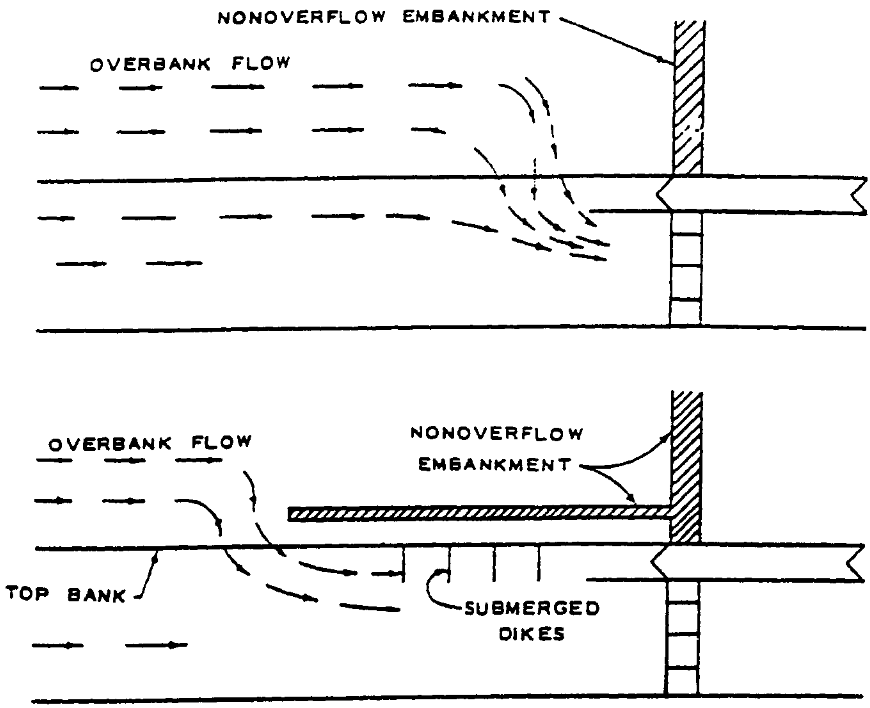


FIGURE 9-11. Effect of Overbank Flow and Location of Submerged Dikes.

required to overcome the effects of these currents. This distance would have to be two to more than four tow lengths upstream of the end of the guard or guide wall, depending on the amount of overbank flow. The fill or dike along the adjacent bank would also provide a guide for tows in determining channel limits. The top of the fill or dike would have to be at least as high as the elevation of the maximum navigable flow for maximum effectiveness.

9.8 LOWER LOCK APPROACH

9.8.1 Currents Affecting Navigation

Currents affecting navigation in the lower lock approach depend on channel alignment; flow from spillway, lock-emptying outlet, and powerhouse; and flow from or toward the overbank. Eddies forming in the lower lock approach and flow moving from the spillway toward the adjacent bank produce currents that could be objectionable to navigation attempting to approach the lower guide or guard wall (Figure 9-12). Bends in the channel a short distance downstream of the approach could increase the intensity of these currents and their effects on the movement of tows. A straight channel extending downstream of the lock would tend to provide better navigation conditions for tows approaching and leaving the lock. Eddies in the lower approach affect upbound tows because of the upstream currents along the bank and riverward currents at the upstream end of the eddy.

9.8.2 Guide and Guard Walls

Guide or guard walls are used to assist upbound tows in approaching and becoming aligned for entrance into the lock. The selection of the type of wall depends on foundation conditions and currents that could be expected with the completed structure. Use of a guard wall instead of a guide wall would tend to facilitate the movement of tows approaching the wall since the outdraft currents would tend to move the tow toward the wall instead of away from the wall. A long guard wall could increase the head on the lower lock gates at the conclusion of a lock-emptying operation when the

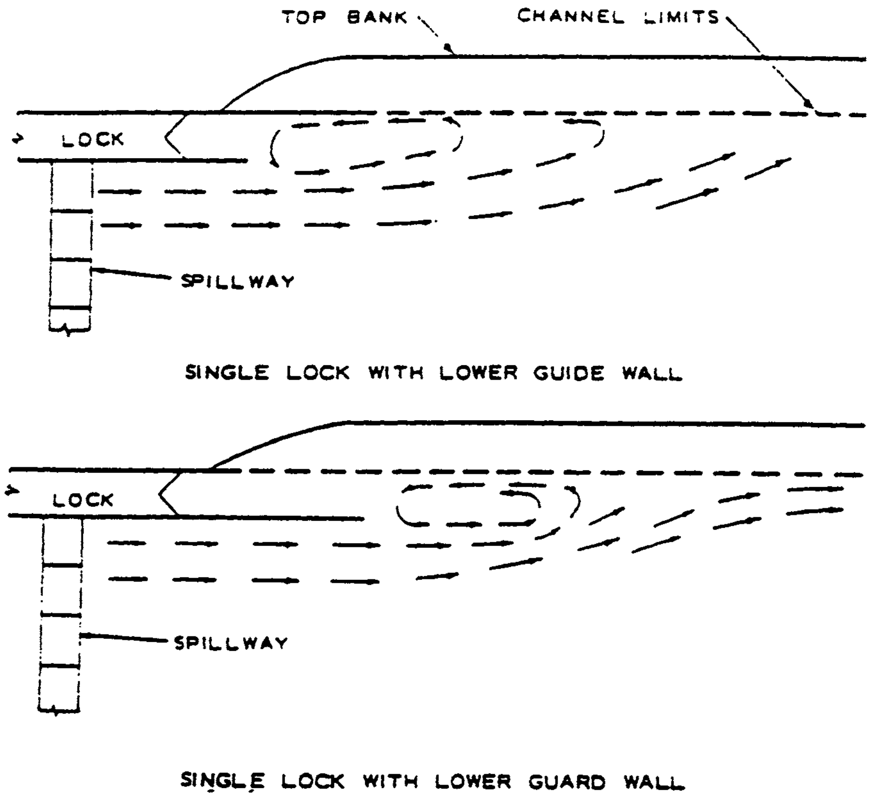


FIGURE 9-12. Currents in Lower Lock Approach.

emptying outlet is on the river side of the lock. Model studies have indicated that installation of ports in the lower guard wall would, in most cases, have little or no effect on the eddy in the lower approach or on the head on the lower lock gate. Where the channel bends riverward of the approach, currents moving from the spillway toward the adjacent bank would tend to resist the movement of the head of a downbound tow in the direction of the bend in the channel. In this case, a guard wall with sufficient channel width landward of the wall would permit the tow to move its stern landward and turn riverward before being seriously affected by the currents. Where the channel bends in the direction of the land side of the lock, the effects of the spillway currents are reduced. However, in this case, sight distance would also be reduced for tows approaching along that side of the channel.

9.8.3 Overbank Flow

Low overbank areas or low dikes forming the lockside bank line can create serious crosscurrents when there is substantial flow toward the overbank or over the dikes (Figure 9-13). These currents would tend to move the tow landward or resist the movement of downbound tows riverward. This condition can be alleviated by erecting a nonoverflow fill or dike along the top bank extending downstream of the end of the lock wall or by raising the elevation of the dikes within the channel to prevent any substantial flow over the dikes. Flow over the overbank or over the dikes would increase any shoaling tendencies existing in the channel along the bank or dikes.

9.9 SHOALING IN LOCK APPROACHES

9.9.1 Upper Lock Approach

In streams carrying sediment, shoaling in the approaches could be a factor and should be considered in the selection of a site for the lock and dam

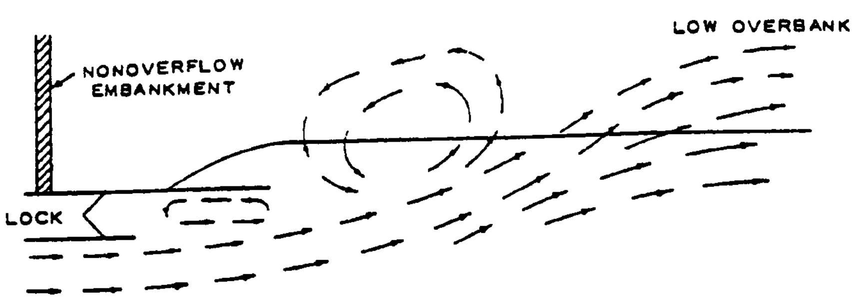


FIGURE 9-13. Low Overbank Adjacent to Lock Approach.

structure. The movement of sediment and formation of bars in the reach considered for the structures should indicate if a shoaling problem exists in the upper approach. In bends, a natural tendency exists for sediment to be moved away from the concave bank; but in relatively straight reaches, special treatment utilizing training structures might be required. Increasing flow toward the lock side of the channel to maintain depths would increase velocities and crosscurrents. By using a guard wall with ports, the crosscurrents can be reduced. Since the tops of ports must be below the bottom of the depth of loaded barges, velocities of the bottom currents are increased, reducing the tendency for shoaling in the approach to the lock landward of the guard wall.

9.9.2 Lower Lock Approach

Shoaling in the lower lock approach is a problem encountered at most structures located in sediment-carrying streams. The seriousness of the problem increases with the amount of sediment moving in the stream and will vary depending on the characteristics of the stream, particularly as affected by variations in discharge. Some of the sediment moving along the lock side of the channel is moved into and deposited in the lower lock approach channel where velocities are reduced by the sudden increase in channel width and depth downstream of the lock. Some deposition is also caused by the action of the eddy formed in the approach.

Normally, in streams carrying a heavy sediment load, the channel is wider at the dam than farther downstream to provide for pier widths, elevation of the gate sill, and capacity of the spillway to pass uncontrolled flows with little increase in stage. Due to the greater channel width, some of the sediment moving through the dam during the higher flows could be deposited between the dam and the lower end of the lock wall. Because of the shallow depth and steeper slope in that reach, this material could then be moved downstream into the lock approach, even during controlled riverflows.

Shoaling in the lower lock approach can be a serious, continuous problem unless remedied. Dredging in the approach would be costly and would interfere with traffic using the lock. In streams carrying a heavy sediment load, periodic dredging could be required since any shoaling on the approach side of the lock wall would make it difficult for tows to approach the wall and become aligned for entrance into the lock.

9.9.3 Reducing Shoaling in Lower Approach

Model studies indicated that a properly designed wing dike extending from the end of the riverward lock wall and angled slightly riverward would reduce the frequency and, in some cases, the amount of dredging.

Experiences with the Arkansas River, which carries a heavy sediment load, indicated that these structures have reduced considerably the frequency and amount of dredging and, in some cases, have almost completely eliminated the shoaling problem. The wing dike is designed to permit sediment-free surface flow to move over the top of the dike and thus prevent or reduce the amount of sediment-laden bottom current that would move into the approach channel around the end of the dike. The dike must be high enough above the bed to prevent sediment from moving over the top and low enough to permit sufficient surface flow over the top to prevent the bottom currents from moving landward around the end of the dike. The wing dikes on the Arkansas River were about 400 to 600 ft long, angled about 10 degrees riverward of the lock wall with crest 2 to 3 ft above normal lower pool elevation (Figure 9-14).

Reduced shoaling in the lower approach channel could be accomplished by opening the gates near the lock wider than those on the opposite side. This operation would cause some of the sediment deposited downstream of the dam to be moved toward the side of the closed or partially closed gates. Since little or no sediment is moved through the dam during controlled riverflows, the effectiveness of this plan would depend on the amount of deposition existing downstream of the dam during these flows. However, caution should be exercised during sediment flushing with this method to avoid scour which could undermine the riverward lock wall. Limited studies in movable-bed models indicated that bypassing flow into the lower lock approach through filling-and-emptying culverts, through the lock, or through a special bypass channel would reduce the tendency for shoaling, depending on the amount of flow introduced into the approach. This, however, would not be practical in removing shoals because of (a) the amount of water required to produce scouring velocities and (b) the dispersion of flow a short distance downstream of the lock walls.

9.10 LOCK-EMPTYING OUTLETS

9.10.1 Location of Outlets

The location of the lock-emptying outlet can affect upbound tows approaching the lock. The discharge during a lock-emptying operation with the emptying outlet located within the lock approach can create currents that could adversely affect tows approaching the guide or guard walls, depending on the location of the tow with respect to the outlet and wall and the discharge from the outlet. Since the currents would not be constant in intensity or direction, they could cause accidents or delays during the approach.

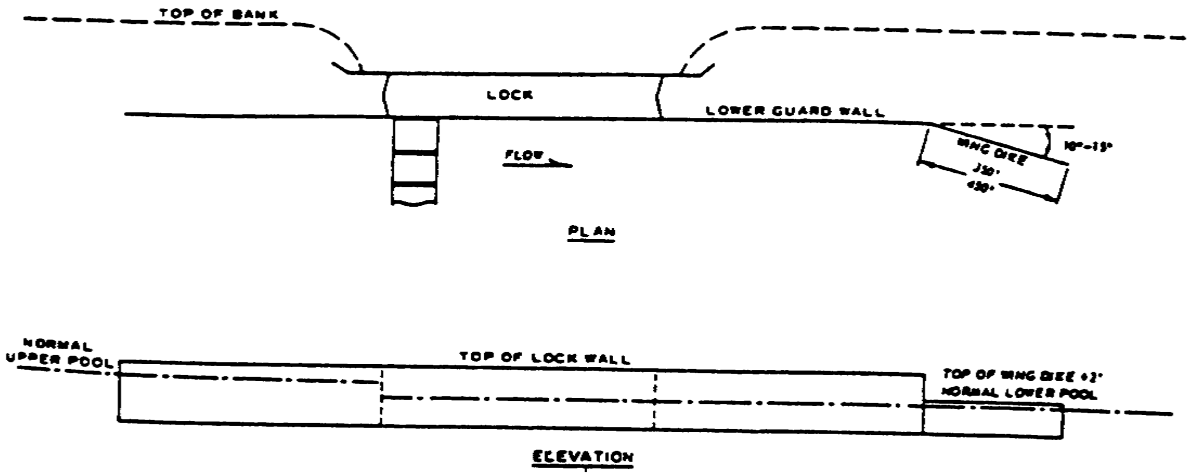


FIGURE 9-14. Wing Dike Used to Minimize Shoaling in Lower Lock Approach.

9.10.2 River Side of Locks

Location of the emptying outlet on the river side of the lock would generally have little or no effect on navigation approaching the lock, particularly when there is some flow through the dam gates near the lock. With the outlet placed on the river side of the lock, water-surface elevations at the emptying outlet could be higher than in the lower approach on the downstream side of the lock gates and could be sufficient to cause some difficulty in the opening of the lower lock gates, particularly during high flows. This difference in elevation would depend on the velocities along the lock side of the channel, water-surface slope along the lock, and the distance from the emptying outlet to the end of the riverside lock wall. The head on the lower gates at the conclusion of the lock-emptying operation can be reduced by placing a wall or rock dike along the upstream side of the lock-emptying outlet. The reduction required would depend on the velocity of currents passing over the outlet and the height of the wall or dike. Since the momentum of the flow past the ports would permit little or no flow through the ports, ports in the lower guard wall would, in most cases, have little effect in reducing the head on the lower lock gates. Generally, sediment deposition into and over the emptying outlet basin would have little effect on lock emptying. Limited tests have indicated that with a head differential (inside and outside the lock) of at least 10 ft greater than the height of the fill over the outlet, the fill material consisting mostly of clean sand would be removed by the emptying operation with little or no effect on lock-emptying time.

9.10.3 Outlets on Land Side and River Side of Lock

Emptying systems with discharges from the land side of the landward lock wall and river side of the riverward wall could be provided when there are adequate water areas on both sides of the lock. Such areas are available with some arrangements, particularly with U-frame type locks. Some reduction in construction cost would be realized with this emptying system since less lock wall would be required downstream of the lower gates than with the inside wall discharge. Deposition over the lock-emptying outlet could affect lock-emptying time and should be considered, particularly with outlets on the river side of the locks.

9.10.4 Surges in Canals

Locks with emptying systems discharging in navigation canals often require special provisions to reduce the magnitude of surges. The Bay Springs Lock on the Tennessee–Tombigbee Waterway uses a ported diffuser manifold that spreads the discharge uniformly across the downstream channel. Other canal projects have used off-channel surge basins that slowly return the discharge water to the canal. This system usually requires a special bypass to allow the last few feet of water in the chamber to flow directly to the canal, providing a more rapid equalization of the water level inside and outside the lock without producing excessive surges. Surges can also be reduced by increasing lock-emptying time with slower valve opening.

9.10.5 Filling from and Emptying into Adjacent River Channel

Locks in bypass canals with the dam in the adjacent river channel could have lock-filling intakes and lock-emptying outlets in the river channel to reduce surges. However, intakes in the river would not fill the lock to the same elevation as that of the water surface in the canal, and special arrangements would be required to either open the lock gates against a higher head or to fill the remainder of the lock from the canal. Discharges into the river channel would not permit the lock to empty to the same level as the water level in the canal downstream of the lock. In this case, a special bypass would be required to empty the remaining portion of the lock into the canal or to open the lower gates against a higher head.

9.11 HYDROPLANTS

9.11.1 Effects of Operation

With the high cost of fuel, the use of hydropower plants in connection with navigation locks and dams has become more attractive and economically feasible and should be considered in the design of the project. Hydroplant operation, when included as part of the navigation project, can produce surges and currents that could be objectionable or hazardous for navigation. To reduce the effect of powerhouse operation, the power facilities should be on the side of the stream opposite the lock or locks when practicable. With this arrangement, power releases usually have little effect on navigation conditions when there is substantial flow through the spillway. With little or no flow through the spillway, flow from the powerhouse tailrace could produce adverse currents in the lower lock approach. Conditions could be particularly objectionable during startup and shutdown because of the surges created. The alignment of the powerhouse should be such that the discharge is not directed toward the lower lock approach. With powerhouse releases and no flow through the dam, a large eddy would form that could extend across the channel to below the lower end of the lock wall, producing currents moving back upstream toward and along the gated spillway back to the tailrace (Figure 9-15). The currents could also be hazardous for small boats navigating or drifting below the spillway since these currents

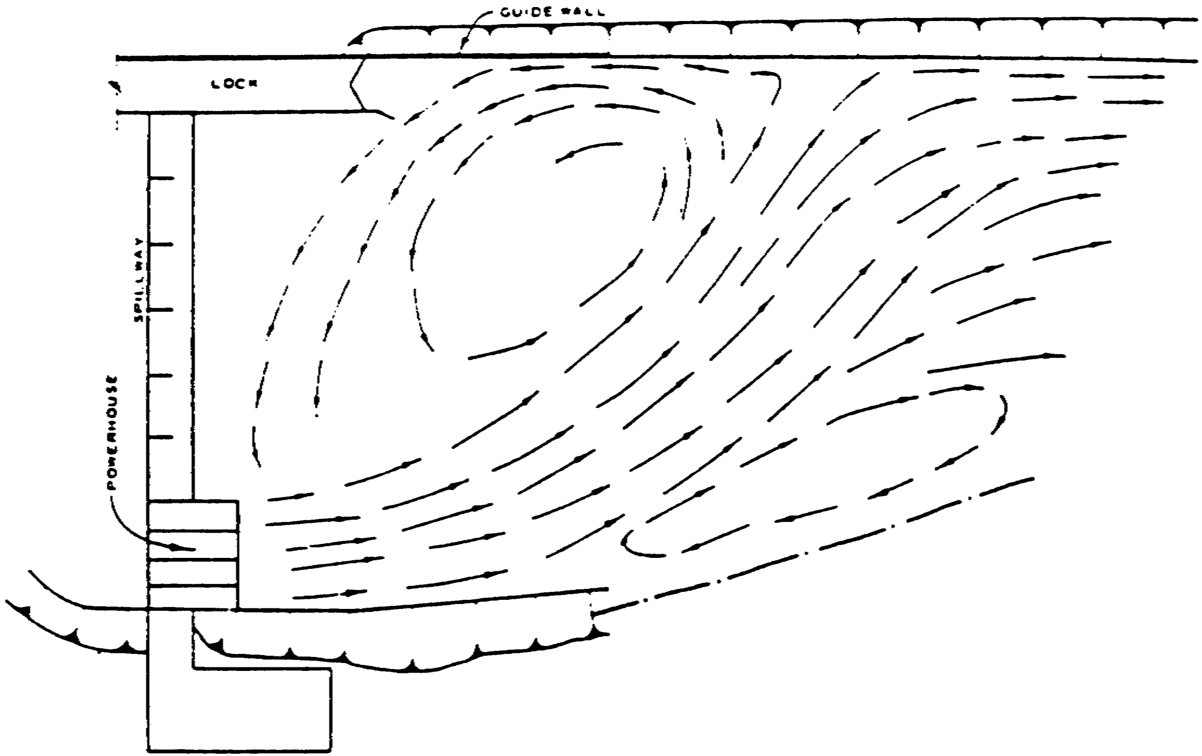


FIGURE 9-15. Effect of Powerhouse Releases on Currents in Lower Lock Approach.

would tend to move them into the tailrace; this condition can be eliminated or minimized by means of a splitter wall between the powerhouse and spillway as shown in Figure 9-16. The eddy currents could be strong enough to move any sediment deposited in the area toward and into the lock approach channel. The effect of the eddy on navigation and on conditions in the lower approach can sometimes be eliminated by permitting some flow through the second or third gate from the lock. A flow equivalent to 20 to 25% of the powerhouse discharge has been adequate in some cases. The amount of flow needed to offset the effects of powerhouse releases will vary with each structure layout and channel configuration.

9.11.2 Reduction of Adverse Currents

The effect of the eddy on navigation can be reduced or eliminated by using a guard wall (river side) in place of a guide wall or by using a long rock dike angled riverward on the river side of a lock with a guide wall (Figure 9-16). In most cases the elevation of the dike needs to be only as high as the tailwater elevation with maximum powerhouse flow and no flow through the spillway. Surges in water level through the upper and lower pools during starting and stopping of powerhouse releases could create problems when there is little or no riverflow. The effect of hinged pool operation during intermittent powerhouse operation and the regulation of the rate of starting and stopping of units should be considered.

9.12 SOURCE

The majority of the information in this chapter came from EM 1110-2-1611, *Layout and Design of Shallow-Draft Waterways*.

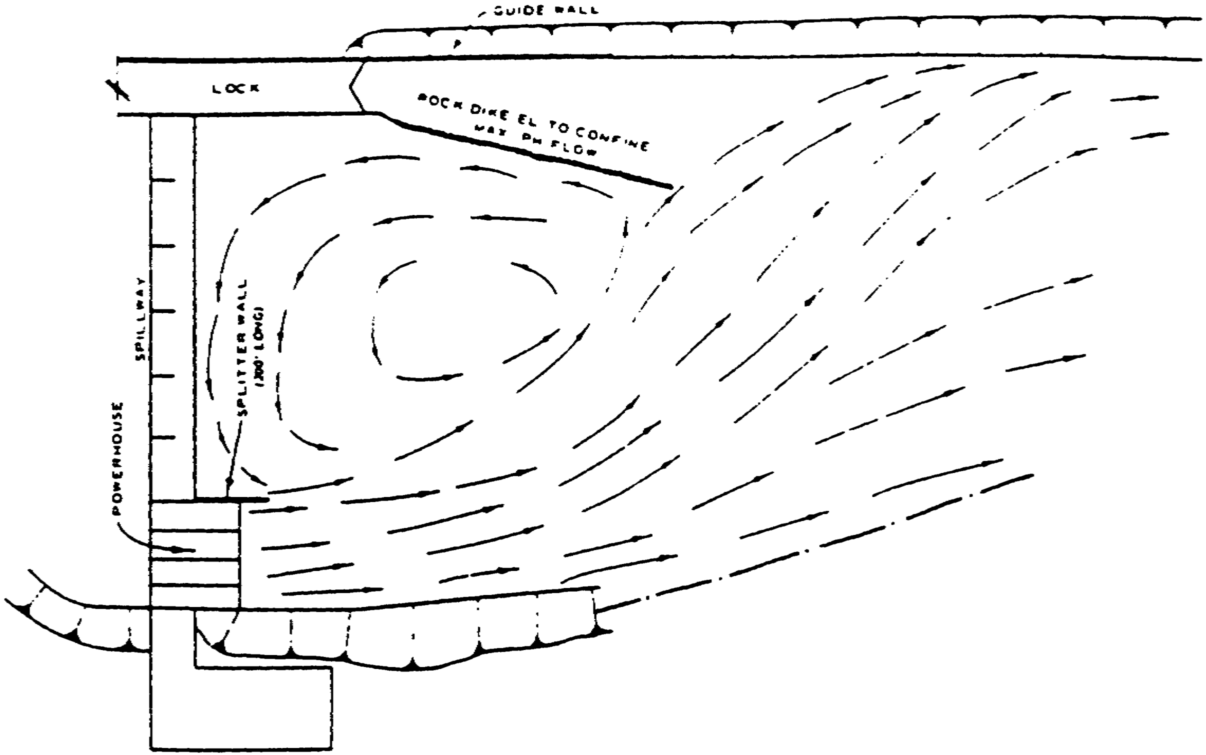


FIGURE 9-16. Dike Used to Minimize Effect of Powerhouse Releases on Navigation in Lower Lock Approach.

Chapter 10

LOCK DESIGN

10.1 PROJECT FUNCTION

10.1.1 Primary Components

All lock designs presented in this Manual contain the four primary components given in the following and shown schematically in Figure 10-1.

10.1.1.1 Upper Approach. The canal immediately upstream from the lock is referred to as the upper approach. The guide wall serves to align and to guide a downbound tow into the lock chamber and is usually a prolongation of one wall of the chamber. The guard wall provides a barrier that prevents the tow from entering an area having hazardous currents or potentially damagable or damaging structures. The term guide-and-guard wall may be used when the combination of functions results in deviations from usual guide wall design practice. Guidelines for approach channel design are included in EM 1110-2-1611.

10.1.1.2 Lock Chamber. The downbound traffic is lowered to the lower pool and the upbound traffic is raised to upper pool within the lock chamber. The upper and lower gates are movable barriers that can be opened to permit a vessel to enter or exit the chamber. Sills, which extend across the lock chamber at the base of the gates, provide a surface for gate closure and are the structural limits for navigable depth in the lock. Lock wall appurtenances are recessed so that the clear width and the usable width are identical. Conversely, because of clearances provided for gate operation and for longitudinal tow drift, the usable length of the chamber differs from the commonly specified nominal lengths, that is, less than the pintle-to-pintle length shown in Figure 10-1. The difference between upper and lower pool elevations is termed lift.

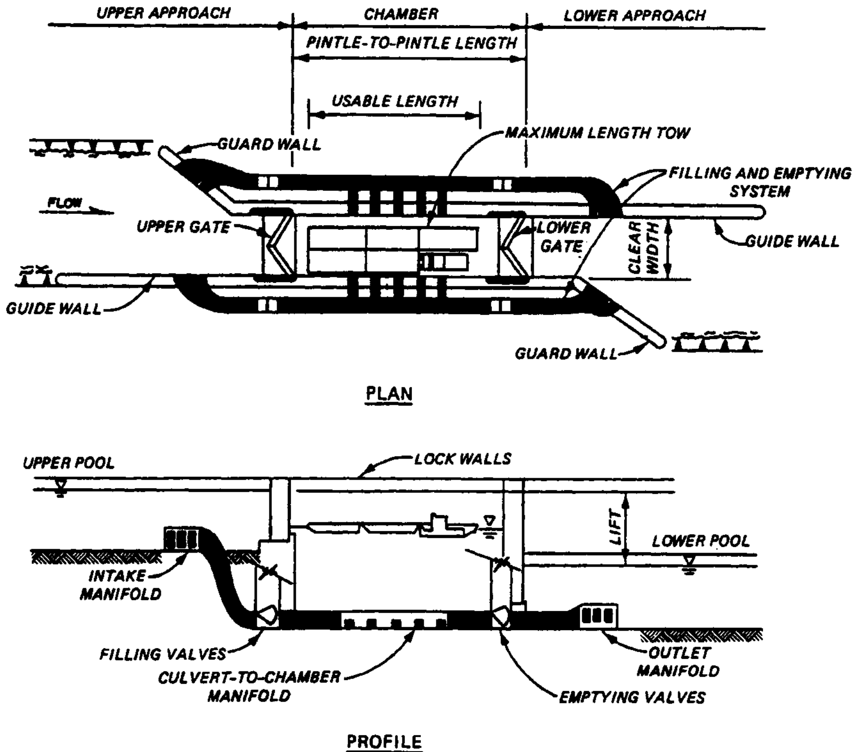


FIGURE 10-1. Common Lock Features for a Lock with Culverts in the Sidewall.

10.1.1.3 Filling-and-Emptying System. For a lock-filling operation, the emptying valves are closed. The filling valves are opened. Flow enters the intake manifolds and exits by means of the culvert-to-chamber manifolds into the lock chamber. For emptying, the filling valves are closed and the emptying valves are opened. Flow enters the culvert-to-chamber manifolds and exits by means of outlet manifolds. Many differences are possible and the idealized system and an actual design are shown in Figure 10-1.

10.1.1.4 Lower Approach. The canal immediately downstream from the chamber is referred to as the lower approach. Guide, guard, and guide-and-guard walls are used and defined similarly both upstream and downstream from the lock (EM 1110-2-1611).

10.1.2 Special Needs

Operation and maintenance considerations (as well as more site-specific topics such as environment, relocation, and geotechnical factors) require additions to the schematized navigation lock shown in Figure 10-1. Construction cofferdams, emergency closure devices, surge suppression pools, and impact barriers are examples of more common special needs that are studied during hydraulic design of navigation locks.

10.1.3 Classification Systems

Two methods are used to classify lock projects.

10.1.3.1 Project Classification (Lift). A lock lift is used to classify the lock project in one of four categories. The categories are listed in Table 10-1.

10.1.3.2 Design Classification (Filling-and-Emptying Systems). Specifications regarding within-chamber manifolds, baffles, and other structural elements are derived from laboratory testing and prototype experience. Small variations in these elements, particularly for high-lift locks, may cause significant surface currents or local turbulence unfavorable to lock operation. Two specific design alternatives are suggested in this manual for each range of project lifts. Schematics of the suggested designs are shown in Figure 10-2 and comments regarding their applicability are included in Table 10-1. Higher-lift designs function well at lower lifts; however, increased costs are also associated with higher-lift designs.

10.1.4 Chamber Performance

During hydraulic design, meeting the project capacity economic constraint requires reducing the time, termed operation time, required to fill or empty the chamber to a value equal to or less than the value used for project authorization. The within-chamber navigation constraint on rapid filling is termed chamber performance; acceptable chamber performance is normally studied by means of filling-and-emptying operations in small-scale physical hydraulic models. Typical observations are as follows.

10.1.4.1 Surface Currents and Turbulence. Acceptable performance requires that surface turbulence hazardous to small vessels be identified and minimized.

10.1.4.2 Drift of Free Tows. The movement of unmoored vessels (from the traffic mix) must be acceptable to navigation and lock operations and not be hazardous to either vessels or structure.

10.1.4.3 Hawser Forces. Mooring line stresses required to restrain the vessel from longitudinal and lateral movement must be acceptable to navigation and to structural design. Specific numerical limiting values have been placed on model hawser stresses. The historic development is based on

TABLE 10-1. Classification of Projects by Lifts

Range of Maximum Design Lift (ft to ft)	Project Classification	Corps Locks (%)	Suitable Design Types
0 to 10	Very low	25	End filling-and-emptying systems are lift suitable. Each of the three general types (gate, valve(s)-in-gate, and loop culvert) can normally provide satisfactory chamber conditions. Choice of type is influenced by economic, operational, and layout factors. The sector gate has been used exclusively for CE very-low-lift designs since 1950.
10 to 30/40	Low lift	60	Wall culverts with side ports (side-port systems) are generally best suited for lifts below about 30 ft. The auxiliary system using lateral manifolds is suitable for low-lift projects requiring one culvert lock operation. Simplified high-lift designs have been model-tested for lifts in the 30 to 40 ft range.
30/40 to 100	High lift	15	Longitudinal manifold systems are suitable. Choice of type (4 or 8 manifolds) is influenced by economic and layout factors. Recent designs subdivide the flow by means of horizontal rather than vertical piers.
Over 100	Very high lift	< 1	These projects are outside the range of CE lock operational experience; the exception is John Day Lock (107 ft lift) on the Columbia River.

breaking strength of one used 2.5 in. diameter manila hawser: a 10,000 pound loading has been used as a safe nonbreaking value. Many years of prototype observation and model testing have shown that when a lock is designed not to exceed the hawser stresses given in (1)–(3) below will be satisfactory for the design vessel as well as for small craft:

1. *Barge tows:* for various sizes and numbers of barges in any location in the lock chamber, the hawser stress as extrapolated from a model does not exceed 5 tons.

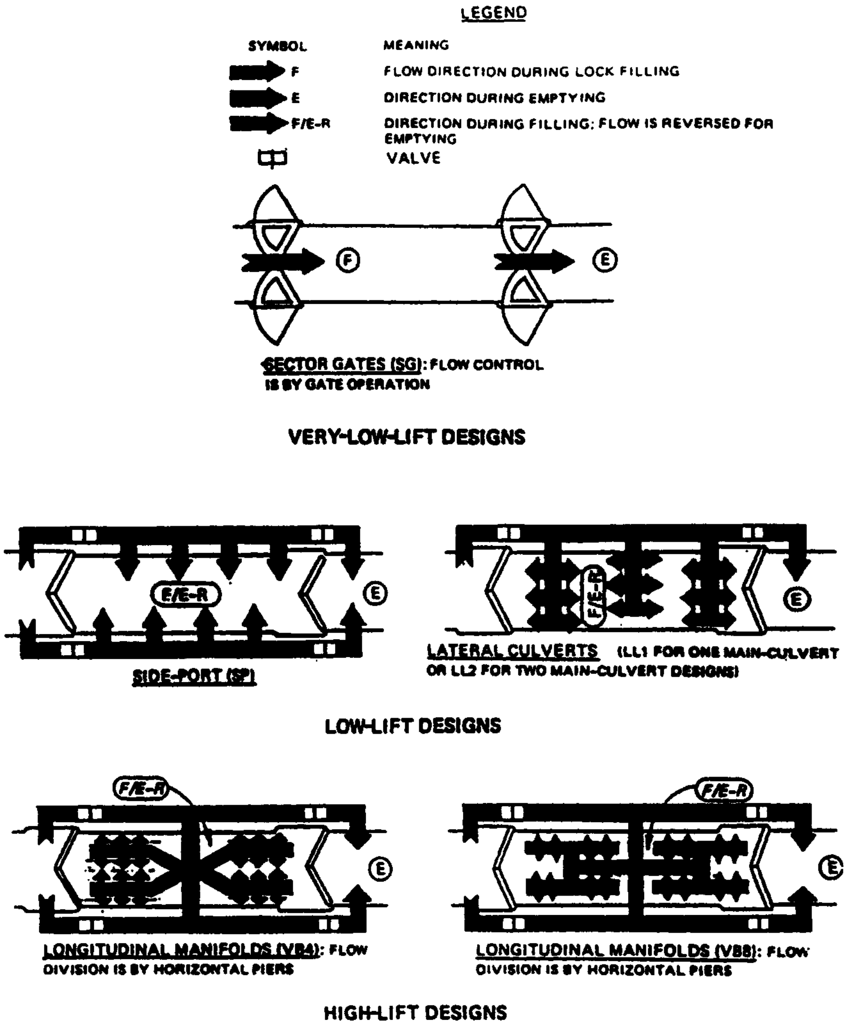


FIGURE 10-2. Flow Distribution of Recommended Designs.

2. *Single vessels—ships up to 50,000 tons:* hawser stress does not exceed 10 tons.
3. *Single vessels greater than 50,000 tons:* hawser stress for larger vessels is allowed to exceed 10 tons, since these vessels require more mooring lines than either barge flotillas or the smaller single vessels. Model tests indicate that if a lock-filling system is designed to meet guidance (1) and (2), hawser stress (extrapolated from the model) will not exceed approximately 25 tons for vessels up to 170,000 dwt.

10.1.5 Application

Time saved during lockage is economically significant at most projects and becomes more important when growth of traffic begins to cause prolonged queuing delays. Decreased operation time causes reduced total transit time unless surges and currents in the approaches adversely affect entry and exit conditions. By means of model and prototype tests and design studies, filling-and-emptying systems have been developed that achieve operation times near 8 min. For existing systems, operation-time benefit, usually presented as a per-minute value, is used to evaluate design modifications that may vary operation time between 8 and 10 min for low-lift and 8 and 12 min for high-lift projects.

10.2 CHAMBER ALTERNATIVES

10.2.1 General

The number and size of chambers are based primarily on capacity studies with system standardization and economics as major constraints. Chamber alternatives are briefly discussed in the following paragraphs; guidance and data relating to navigation facility for both single-chamber and multichamber projects are included in Chapter 9.

10.2.2 Number of Parallel Chambers

In the initial development stage of a waterway transportation system, common practice has been to provide one chamber at each project; then, as traffic has increased, additional chambers have been added. For a new project on a developed waterway, where traffic patterns are well established and continued growth is assured, two or more chambers may be initially justified on an economic basis. A need for continuous operation may lead to double chambers since, in the event of outage of one lock, essential traffic can be handled on a priority basis. In redevelopment of the Ohio River system, a minimum of 2 locks have been provided at each of the 19 locations.

10.2.3 Chamber Dimensions

Chamber dimensions are influenced by sizes of existing barge and towing equipment; conversely, existing barges and towing equipment have been influenced by sizes of existing chambers. Most of the locks built in the United States since 1950 have usable horizontal dimensions of 84 by 600 ft, 110 by 600 ft, and 110 by 1,200 ft. A number of locks with other sizes have been built: 56 by 400 ft; 75 ft width with lengths varying from 400 to 1,275 ft; 80 by 800; 82 by 450 ft; and 84 ft width with lengths of 400, 720, 800, and 1,200 ft. Recent western locks (along the Columbia and Snake Rivers) have usable dimensions of 86 by 675 ft. Additional lock chamber length is provided for clearance between the tow and the gates so that gate-to-gate chamber length is greater than usable length. Smaller chambers are used on waterways where the traffic is exclusively recreational boats and small craft.

10.2.4 Chamber Types

The majority of US lock chambers are for commercial tows with drafts equal to less than 14 ft, 9 ft being the most common. Certain waterways require chambers that are unusual but that provide supplemental operational experience to recent US lock design, testing, and operational data; these chambers are not evaluated herein. The following listing includes five such chambers.

10.2.4.1 Ship Locks. Chambers used by ships are given in Table 10-2.

10.2.4.2 Great Lakes Shipping. Commercial vessels are normally individually powered and relatively (for ships) shallow draft. For example, ships with drafts in the range of 16 to 25 ft and sizes from 15,000 to 30,000 dwt are accommodated on the Great Lakes. Lock entry and exit requirements for these types of vessels differ from either barge tow or oceangoing-ship needs.

10.2.4.3 Deep Drafts. Chambers designed for both large tows and deep-draft ships (draft 25 ft or greater) need special entry and exit features. Sills are located sufficiently deep to accommodate squat, trim, and sinkage. Tow-

TABLE 10-2. Ship Locks

Navigation System	Lock Name	Lock Size	Normal Lower Sill Submergence, (ft)	Normal Lift, (ft)
Gulf Intracoastal Waterway	Inner Harbor	75 × 640	31	9
Lake Washington	Chittendon (Large)	80 × 760	29	26
Ship Canal	Chittendon (Small)	28 × 123	16	26
St. Marys River,	MacArthur	80 × 800	31	22
South Canal	Poe	110 × 1,200	32	22
St. Marys River,	Davis	80 × 1,350	23	22
North Canal	Sabin	80 × 1,350	23	22

ing winches and other assisting mechanisms are used. Ships greater than 100,000 dwt are assisted. A side-port design has been studied for the New Ship Lock, Mississippi River–Gulf Outlet. test results are for a 150 by 1,200 ft lock; maximum normal head = 18.4 ft; vessel draft = 45 ft (ships) and 9 and 12 ft (tows). Deep-draft navigation projects are discussed in the *ASCE Manual and Reports on Engineering Practice No. 80.*

10.2.4.4 Recreational Locks. Locks having usable lengths less than 400 ft are considered recreational locks. Limited small-tow and special commercial vessels also use many of these locks.

10.2.4.5 Repair Facilities. Drydocks and other similar chambers have mechanical and structural elements comparable to lock chambers. Expeditious closure and sealing during unwatering are major design requirements.

10.3 FOUNDATION AND STRUCTURE CONCERNS

10.3.1 Hydraulic Loading

The foundation and structural features establish the stability and durability of the structure. Hydraulic loadings during construction, completion, and operation are a major concern. These loadings, because of magnitude and spatial and temporal variations, are complex and require particularly thorough study and interdisciplinary coordination. For example, static conditions at chamber full as compared to chamber empty are recurring changes in loadings that influence deflections and stability parameters for the foundation, walls, and sills of the chamber. Known extreme conditions, such as exist during inspections, in addition to filling or emptying, cause recurring changes in differential-pressure loading across structural elements. Unusual extreme conditions, such as exist during unusual valve and emergency operation, are also of concern. For high-lift locks, the hydraulic design includes high-velocity flow so that passageways may require, for example, special treatment to avoid surface cavitation and abrasion damage. The need for relief of pore pressure within the foundation or within monolith cracks and joints is dependent on hydraulic conditions. These loadings are discussed in EM 1110-2-2602.

10.3.2 Chamber Structure

Concrete lock structures have been generally reliable and desirable based on engineering and economic considerations. On waterways where traffic is not heavy and at locations on waterways where the lift is very low, sheetpile locks or possibly earth wall locks have sometimes been used.

10.3.2.1 Concrete Lock Structures. The most common lock structure uses concrete gravity walls founded on either piling or rock (EM 1110-2-2002 and EM 1110-2-2602). Culverts, valve shafts, access passageways, and numerous other special-purpose cavities are contained within the wall. Intakes and outlets may also be formed in the wall although at many locks these are located well outside the actual lock chamber. More unusual concrete lock structures are of the buttress-wall type or have rock walls with anchored concrete facing. For these thin-wall designs, the filling-and-emptying system components are essentially separated from the walls. For the two parallel chambers shown in Figure 10-3, a gravity-wall low-lift design, the intermediate wall serves both chambers. A high-lift lock with concrete gravity walls is shown in Figure 10-4. In Figures 10-5 and 10-6 are high-lift designs with thinner concrete walls anchored to natural rock.

10.3.2.2 Sheetpile Structures. Very-low-lift projects permit structures other than concrete to be considered for design; masonry, earth embankment, and sheetpile structures have been used. Sheetpile lock walls are of two basic types: sheetpile cells and M-Z sheetpiling supported laterally by wales and tie rods. Sheetpile locks are filled and emptied by sector gates or other very-low-lift systems. Gate bay monoliths are normally concrete. The low initial cost for sheetpile structures is offset by short useful life and high maintenance. Recent use has been at sites where temporary (or emergency) locks were needed. A sheetpile cellular lock is shown in Figure 10-7. Sheetpile structures are commonly used for cofferdam functions.

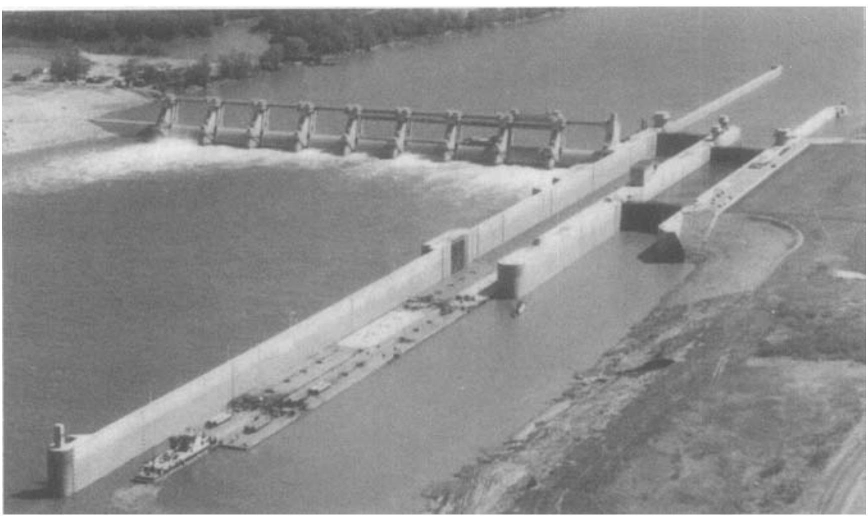


FIGURE 10-3. Parallel Locks with Gravity Walls. Willow Island Locks, Ohio River, with Design Lift of 20 ft.

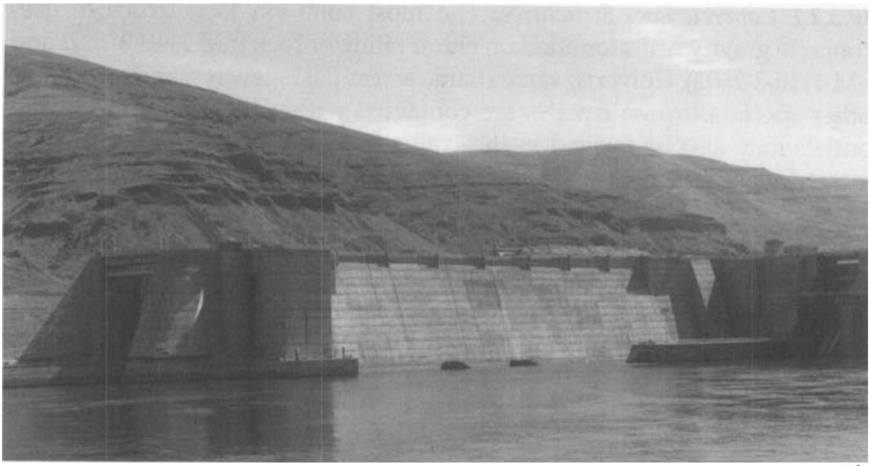


FIGURE 10-4. Lock with Gravity Walls. Lower Granite Locks, Snake River, with Design Lift of 100 ft.

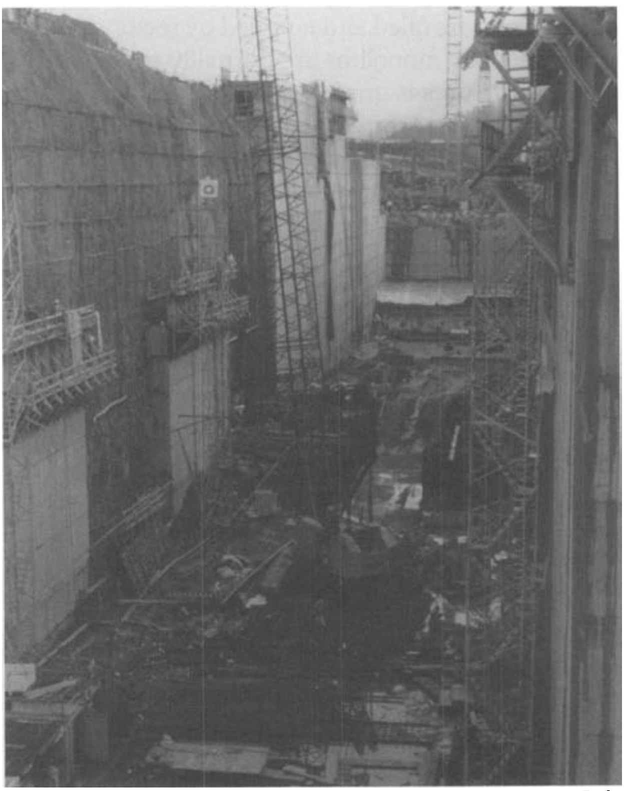


FIGURE 10-5. Lock with Thin Walls. The New Bonneville Lock, Columbia River, with Design Lift 69 of ft.

FIGURE 10-6. *Lock with Thin Walls. Bay Springs Lock, Tennessee–Tombigbee Waterway, with Design Lift of 84 ft.*

FIGURE 10-7. *Temporary Lock with Cellular Sheetpile. Lock and Dam No. 52, Ohio River, with Design Lift of 12 ft.*

10.3.2.3 Earth Embankments. Earth embankments with concrete gate bays are considered for low-use, very-low-lift projects. For example, these locks are included in the Gulf Intracoastal Waterway to prevent saltwater intrusion and to prevent adverse or dangerous currents during abnormal tide conditions. The walls are essentially levees, with riprap protection on the side slopes. Riprap protects the bottom of the channel (the chamber) from scour due to towboat propellers. Tows moor to timber guide walls during lockage. A lock of this type equipped with sector gates is shown in Figure 10-8.

10.3.3 Guide and Guard Walls. Navigation needs (see EM 1110-2-1611, EM 1110-2-1613, and Chapter 9) require the proper location and alignment of guide and guard walls and are resolved by means of general river hydraulic models; project purposes in addition to navigation are normally also of concern. These studies, which require preliminary estimates of lockage inflow and outflow hydrography, also determine the impact on navigation regarding type of wall (i.e., floating, ported, or solid). When navigation needs are

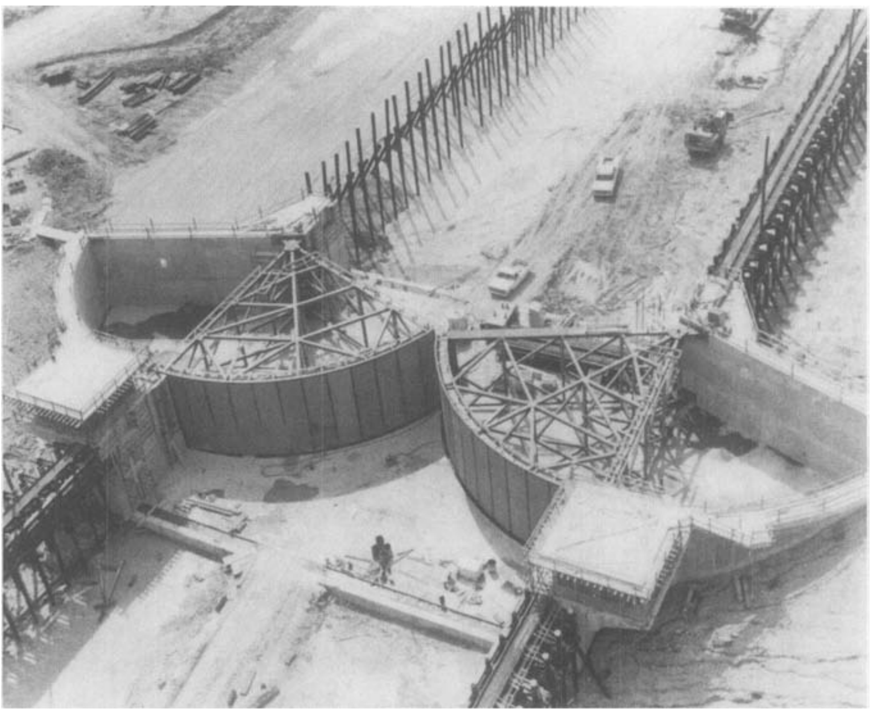


FIGURE 10-8. Earth Embankment with Concrete Gate Bays and Sector Gates. Leland Bowman, Gulf Intercoastal Waterway, with Design Lift of 3 ft.

resolved, then construction and maintenance economics determine the type of wall actually used at a specific project. Similarly, the heights of guide, guard, and lock walls are influenced by operational as well as navigational needs during high river stages. Examples of structural types are:

a. concrete gravity walls,
b. concrete walls supported by structural cellular piling,
c. timber walls supported by pile clusters, and
d. floating moored caisson structures.

Timber structures are normally limited to very-low-lift locks preferably where traffic consists of smaller tows.

10.3.4 Other Structures

Navigation conditions may require mooring facilities, fleeting areas, and other aids. Examples of structures currently in use are pile dikes (Columbia River), pile cluster dolphins, and caissons such as those used for barge docks. Energy absorption required due to barge impact is a design concern.

10.4 FILLING AND EMPTYING

10.4.1 Project Type

Hydraulic design addresses all features relating to filling and emptying the lock chamber. Decisions based on specific authorization requirements that narrow hydraulic options are:

10.4.1.1 Maximum Navigation Lift. This value determines design type as previously shown in Table 10-1. For maximum lift near 10 ft, conservative design practice is to use a low-lift rather than a very-low-lift design type. Similarly, for maximum lift near 40 ft, conservative practice is to use a high-lift rather than a low-lift design type. For low-usage locks or for projects with significant variation in lift, economic considerations want less conservative design. Lifts greater than 100 ft exceed US operating experience.

10.4.1.2 Chamber Navigation Constraints. Project identification studies identify four constraints relative to chambering:

1. vessel characteristics (types, drafts),
2. clear chamber width,
3. usable chamber length, and
4. operation time (economics).

These constraints, compared with existing lock data, establish design status compared to US operating experience. Model- and prototype-tested geometries establish status compared to US verifiable laboratory and field experience. An overview of operating conditions for five specific US design types is provided in Table 10-3; traffic is different mixes of commercial tows and recreational vessels.

10.4.2 Design Type

The following designations for types of lock-filling systems are used throughout this Manual

LC = loop culvert(s)
LCSG = loop culvert(s) and sector gate
SG = sector gates
SP = side ports
SPF = side ports with flume
MP = multiport system
BL1 = centered lateral-manifolds; one culvert
BL2 = centered lateral-manifolds, two culverts
BLC = centered lateral-manifolds; high-lift modified
SBLC = split lateral-manifolds
OC = longitudinal centered and ported culvert
HB4 = horizontal flow divider, 4 longitudinal
HB8 = horizontal flow divider, 8 longitudinal
VB4 = vertical flow divider, 4 longitudinal manifolds
VB8 = vertical flow divider, 8 longitudinal manifolds

New projects are compared in terms of lift, chamber geometry, and navigation constraints with existing designs listed in Table 10-3; however, site-specific conditions may require a different design. For each lift category, the design type is judged as matching, modified, or new as follows.

10.4.2.1 Very-Low-Lifts (0–10 ft). For matching sector gate (SG) designs, sill and floor elevations and gate operation schedules are from specific model-tested designs. Modified designs to accommodate small chamber dimension changes (when geometric similarity is essentially retained) can be reliably determined from existing designs. New designs (due to unusual or more stringent navigation constraints, untested end-filling devices, or major changes in chamber dimensions) require laboratory testing and evaluation to determine chamber performance. Low-lift design types (Section 10.4.2.2) are conservative alternatives for very-low-lift projects

TABLE 10-3. Experience with Recommended Designs (Geometries Constructed Since 1950)

Type	No. of Similar Locks	Chamber Clear Width (ft)	Chamber Usable Length (ft)
Very-Low-Lift Designs (Maximum Lift[a] < 10 ft)			
Sector gate (SG)	1	86	600
	1	84	600
	1	75	1200
	1	75	1150
	1	75	800
	1	56	800
	1	45	800
	7	30	90
	7	30	90
Temporary(SPF)	2	110	1200
Total	23		
Low-Lift Designs (Maximum Lift[b] < 30/40 ft)			
Side port (SP)	10	110	1200
	67	110	600
	10	84	600
	22	56	360
Laterals (BL2)	7	110	1200
	6	110	600
	2	84	720
Laterals (BL1)	7	110	600
Total ~~	131		
High-Lift Designs (Maximum Lift > 40 ft); Longitudinal Manifolds			
4-manifold (HB4)	2	110	600
		86	675
8-manifold (HB8)	1[c]	86	675
Total	3		

[a]Lifts greater than 10 ft are experienced at many of these projects.

[b]Lift experienced during actual operations extends up to about 37 ft; commercial traffic is primarily 9 ft draft tows.

[c]Lower Granite Lock became operational in 1975; tows up to 14 ft draft use this project.

10.4.2.2 Low-Lifts (10–30/40 ft). For matching or modified side-port (SP) designs, sill and floor elevations and valve schedules are from design criteria. For two-culvert projects the choice of lateral culverts (BL2) as compared to side ports has been an economic consideration (structural cost, chamber maintenance, and excavation costs are major factors); the side-port system was least-cost for the Robert C. Byrd new main lock (110 by 1,200 ft, 23 ft normal lift). Unfortunately, existing BL2 designs have unfavorable single-culvert projects operating characteristics that tend to preclude their use for new projects For one-culvert projects (auxiliary or alternative locks) a lateral design (BL1) is used. Because of the broad extent of testing and experience with these types of locks, the need for a new design is considered unlikely. However, were a site-specific situation to require more rigid requirements on chamber performance or to require alternate culvert geometries (due to an unusual site-specific constraint, for example) then an alternative design could be justified. The alternate design would probably be similar in concept to the existing high-lift designs and would require extensive laboratory testing and evaluation to determine chamber performance.

10.4.2.3 High-Lifts (30/40–100 ft). For matching balanced flow designs for both four manifolds (HB4) and eight manifolds (HB8), sill and floor elevations and valve schedules are from design criteria. Matching designs must agree in detail; that is, in addition to chamber dimensions, ports, baffles, sills, and the like, are to be sized and shaped according to either HB4 or HB8 existing details. The complete culvert-to-chamber (crossover culvert) system must also match in geometric detail. Any change constitutes a modified design which, as for a new high-lift design, requires laboratory testing and evaluation in terms of chamber performance and of reliability and durability of the total design.

10.4.3 Lateral Culverts

Concepts similar to the BL2 design have been tested and are in operation at numerous projects. Unlike side-port designs, inconsistency in geometric detail for lateral-culvert designs precludes the development of broad design criteria. The following factors have caused lateral culverts (including the BL2 design) to be viewed as less acceptable than side-port systems (for low-lift) or longitudinal systems (for high-lift).

10.4.3.1 Slow Valving. Four minute or greater valve times have been used extensively; rapid operation requires more rapid valving.

10.4.3.2 Rigid Valve Times. The valve time established during testing (Section 10.4.3.1) cannot be reduced without (Section 10.4.2.1) significant deterioration in chamber performance.

10.4.3.3 Harmonic Oscillations. Natural oscillations of the chamber water surface appear to be excessively stimulated, leading to large hawser forces.

10.4.3.4 Synchronous Valving. Any valving other than two-valve fully synchronized valving causes chamber performance to severely deteriorate in terms of oscillations (Section 10.4.2.3) and free tow movement.

10.4.4 Features

The design considers each of the following six compatible systems.

10.4.4.1 Intake System. Conditions in the upper approach channel are concurrently resolved by hydraulic design, navigation facility and safety, operations, and other multipurpose or multidiscipline concerns. Guide and guard walls are specific items of major concern to navigation. Intake manifold, trash rack, and transition conduit are hydraulic design features.

10.4.4.2 Filling Valve System. Valve design is a hydraulic concern as are the valve well, bulkheads, air vent, and flow-passage designs. Hydraulic loadings required for structural and mechanical detail design are required in addition to flow parameters needed solely for lock filling and emptying.

10.4.4.3 Culvert-to-Chamber System. The culvert, manifold(s), ports, and transitions are hydraulic design features. Chamber navigation conditions (expressed as turbulence, hawser stress, and vessel drift) are highly influenced by culvert-to-chamber geometry.

10.4.4.4 Chamber System. Features making up the lock chamber, such as the upper and lower gates and navigation and operation aids, are concurrently resolved by hydraulic design, navigation facility and safety, operations, and other design functions. The lock sill and chamber floor elevations, manifold recesses, and baffles are hydraulic features.

10.4.4.5 Emptying Valve System. The listing of features is the same as for the filling valve (see Section 10.4.4.2).

10.4.4.6 Outlet System. Conditions within the lower approach channel are, as for the upper approach, multipurpose and multidiscipline concerns. The transition conduit and outlet manifold and baffles and energy dissipator are hydraulic design features. The features within each system are modified during design for each site-specific lock. The systems for each basic design type (very-low-lift, low lift, and high-lift locks) are distinctly different; and within each design type, certain features are varied when necessary to resolve project constraints.

10.4.5 Recent Designs

Projects of each of the seven design types listed in Table 10-3 have recently been designed. Each of the types and the corresponding feature locations are summarized in Table 10-4 and shown in Figures 10-9 through 10-15.

10.5 APPURTENANT CONCERNS

10.5.1 Navigation Aids

These devices are recessed into the lock wall, flush mounted on the wall face, or located on the upper surface of the wall. The objective is to provide assistance to navigation (for all anticipated vessel types) commensurate with clear chamber width and minimum maintenance. Examples are floating mooring bits, ladders, line hooks, check posts, ring bolts, and staff gauges.

10.5.2 Surge Reduction

Currents and water-surface elevations in the upper and lower approaches to the chamber are major concerns to navigation. For canals and smaller waterways these surge effects, during both filling and emptying, are severe constraints to hydraulic design (EM 1110-2-1606). Coordination involving both navigation (EM 1110-2-1611) and hydraulic studies is needed

TABLE 10-4. Design Types and Example Project Locations

Figure No.	Design Type	Design Symbol Type	Project Lock
10-9	SG	Sector Gate	Leland Bowman; Gulf Intracoastal Waterway
10-10	SPF	Side Port/Flume	Lock and Dam No. 52, Ohio River
10-11	SP	Side Port	Ozark; Arkansas River
10-12	BL2	Bottom Lateral (2 culverts)	Belleville Main Lock; Ohio River
10-13	BL1	Bottom Lateral (1 culvert)	Willow Island Auxiliary; Ohio River
10-14	HB4	Horizontal Split Balanced Flow (4 Manifolds)	Bay Springs; Tennessee–Tombigbee Waterway
10-15	HB8	Horizontal Split Balanced Flow (8 manifolds)	Lower Granite; Snake River

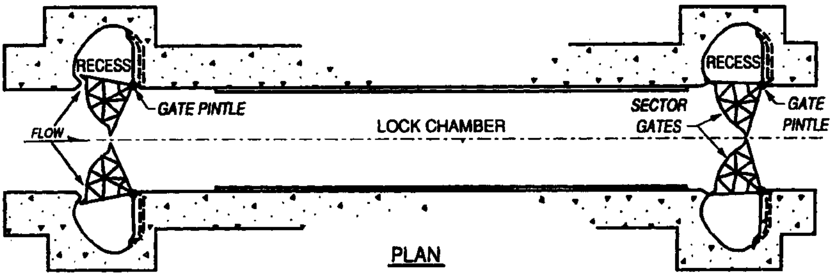


FIGURE 10-9. Sector Gate Lock.

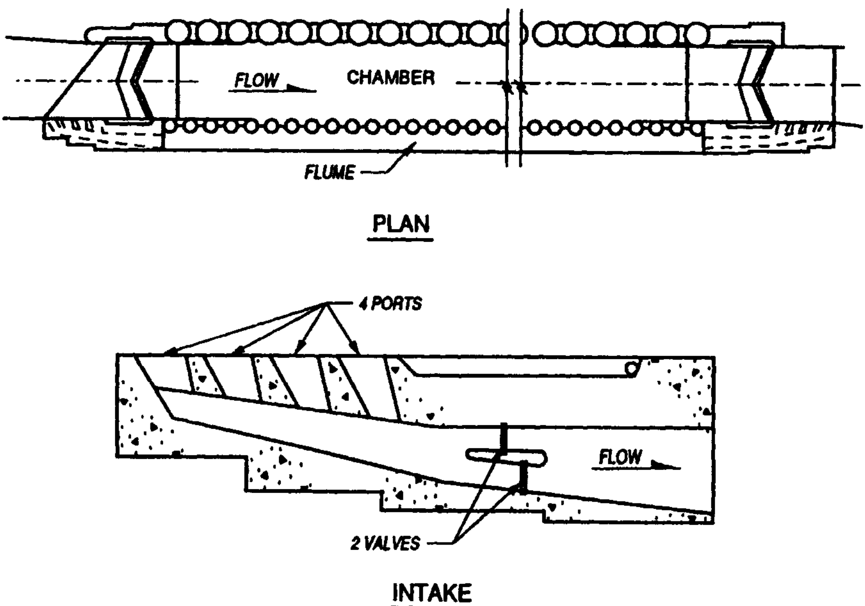


FIGURE 10-10. Side-Port/Flume, Temporary Lock.

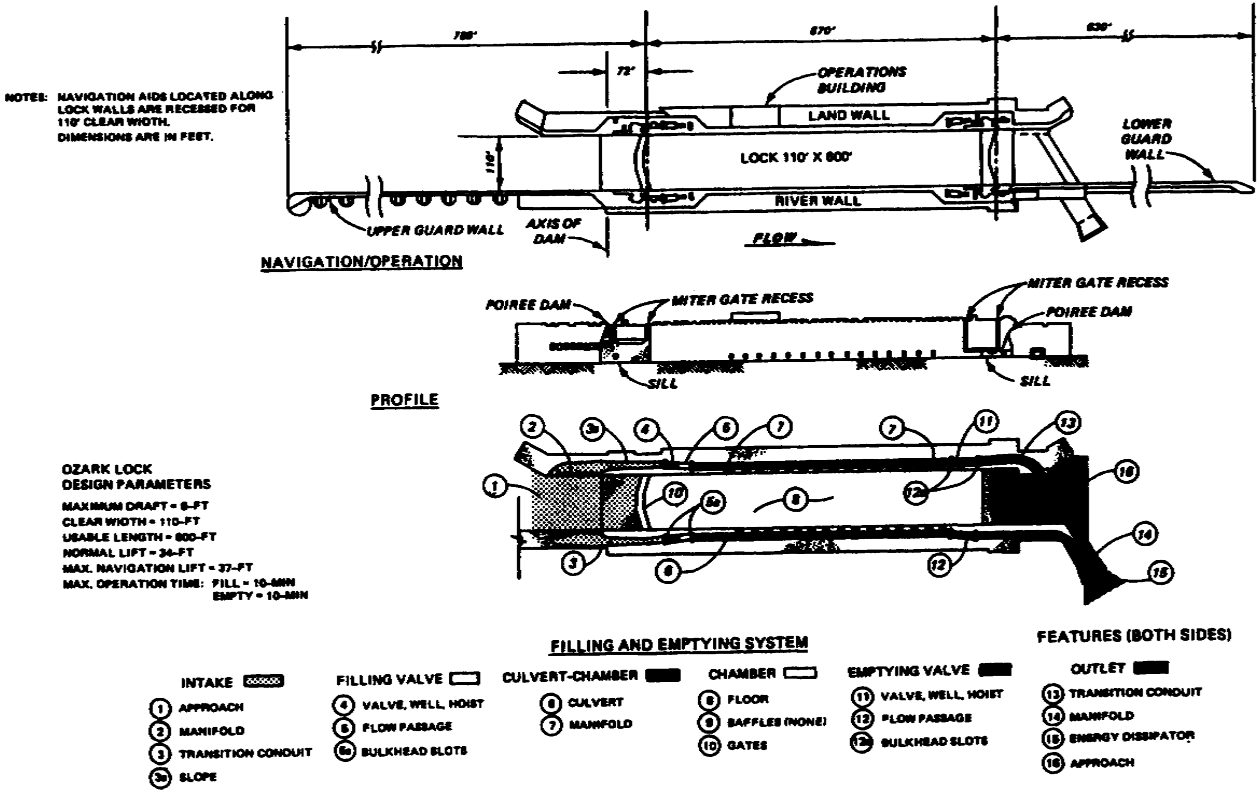


FIGURE 10-11. Side Port Lock, Ozark, Arkansas River.

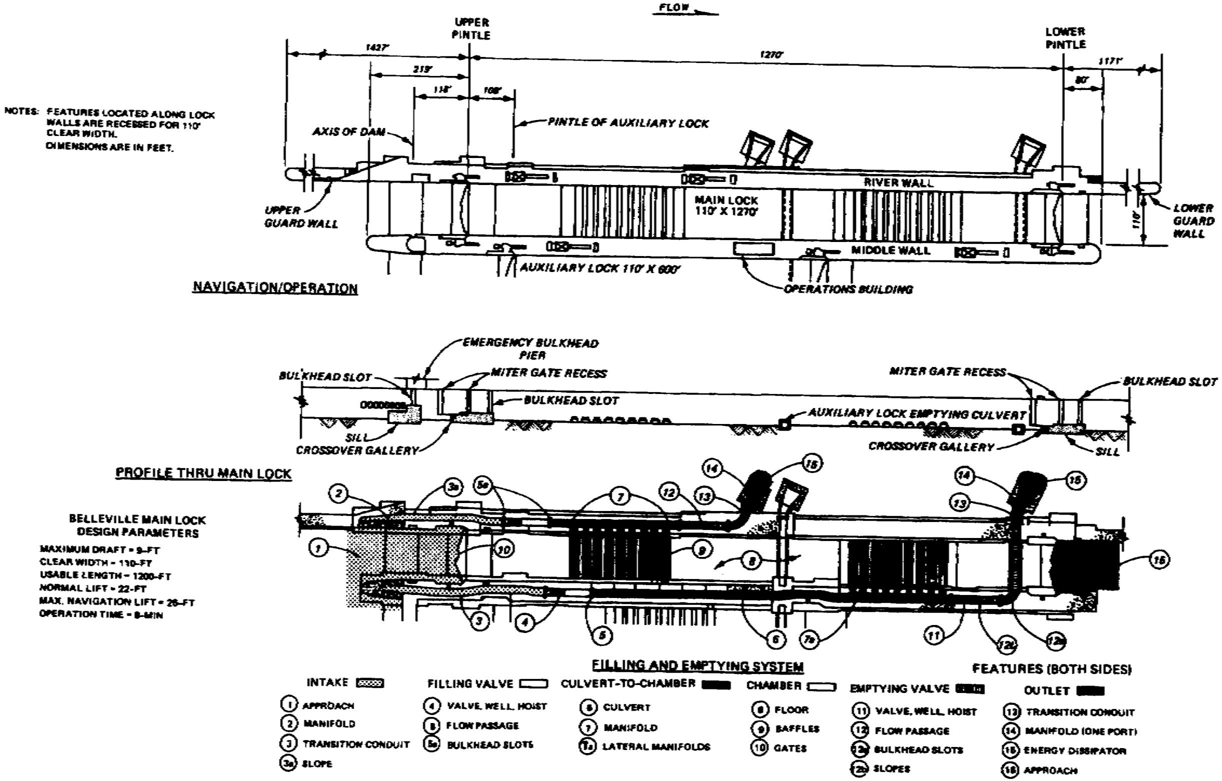


FIGURE 10-12. *Bottom Lateral Lock (2 Culverts) Belleville Main Lock, Ohio River.*

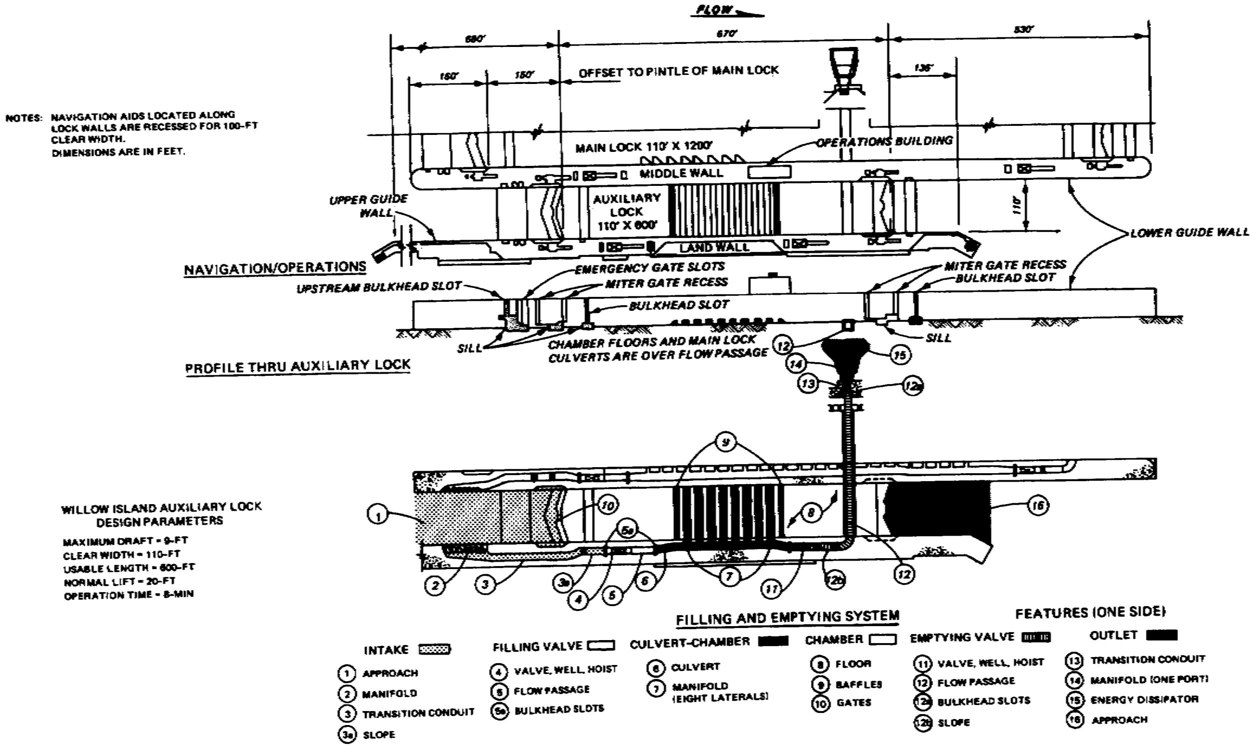


FIGURE 10-13. Bottom Lateral Lock (1 Culvert) Willow Island Auxiliary Lock, Ohio River.

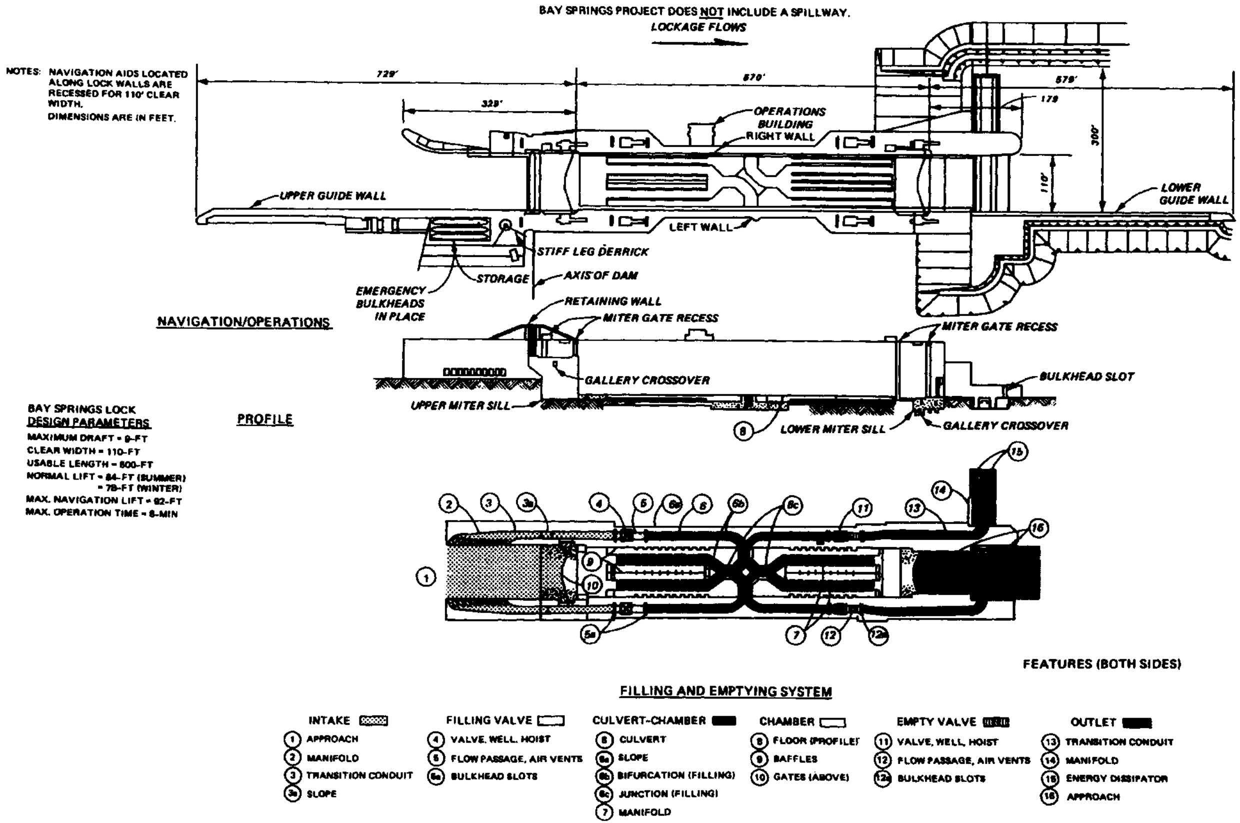


FIGURE 10-14. Horizontal Split Balanced Flow Lock (4 Manifolds) Bay Springs, Tennessee–Tombigbee Waterway.

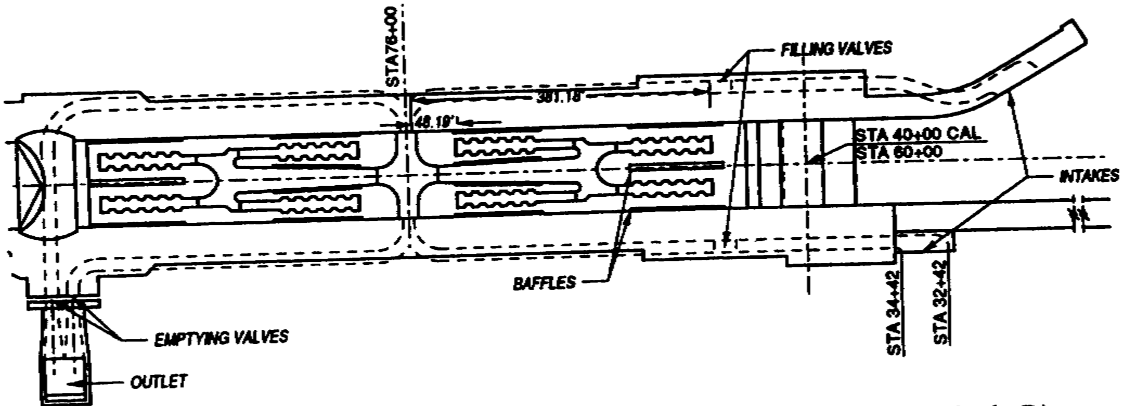


FIGURE 10-15. Horizontal Split Balanced Flow Lock (8 Manifolds) Lower Granite, Snake River.

in order to determine locations of intakes and outlets, alignment and provided types of guide and guard walls, and geometries of the approach canals such that surge effects are acceptable to navigation. In the event that these effects cannot be resolved at acceptable costs, then the hydraulic filling or emptying operation times may be extended either by valving or by using a less efficient hydraulic system. An alternative to slowing the systems is using storage basins (surge reduction basins) adjacent to intakes or outlets.

10.5.3 Impact Barriers

Protection of the upper or lower gates from collision by navigation vessels is the primary objective. Wood, rubber, and metal fenders and bumpers are used on gates, on key locations along guide and guard walls, and on the exposed surfaces of the recessed gates as inexpensive and repairable energy absorbers. Protective equipment is discussed in EM 1110-2-2602.

10.5.4 Water Saving

Environmental or economic factors may require design features directed toward minimizing the quantity of water transferred during lockage. The problem is addressed at these stages in project life:

a. preliminary studies for the selection of the number of chambers and chamber sizes may result in including either a small hydraulic lock or a mechanical lift for smaller (normally recreational) vessels;
b. during design, consideration of either adding an extra set of lower gates (to permit fractional chamber operation) or including a water-saving chamber (to permit saving a fraction of the water normally lost emptying for use during filling) may be warranted. Neither has been feasible for US locks. Staged-lifts normally use less water than single-lift locks at the expense of operating costs and transit time; and
c during operation, lockage procedures directed toward reducing the number of operations required for passing a mix of vessel sizes result in water-savings benefits.

10.5.5 Dewatering

Maintenance is the primary objective. Scheduled inspections require full and partial dewatering of the lock chamber and most flow passages. Provisions to facilitate pumping for elevations below the lower pool should be provided. Closure is during static conditions and is normally accomplished by means of bulkheads. Canal bulkheads above and below the upper and lower chamber gates, respectively, are used to isolate the chamber gates. Culvert bulkheads above and below each valve are used to isolate the culvert valves. Hydraulic design emphasis particularly for high-lift locks, is to shape and locate the culvert bulkhead slots for minimum disturbance to the flow with no cavitation at the boundary while satisfying sealing and structural requirements during closure.

10.5.6 Emergency Closure

Risk associated with failure of the upper miter gates may justify the installation of devices for closure of the chamber during free-surface flow directly over the upper sill. Various closure devices are available as described in EM 1110-2-2703 and EM 1110-2-2602. For a highly developed waterway, such as areas along the middle reaches of the Ohio River, significant monetary losses to commerce and other hazards could result from unrestricted flow. The three principal sources of loss are: loss of pool upstream from the lock, possible flood damage downstream from the lock, and loss to shipping, recreation, and other project purposes in both pools, particularly in the upstream pool.

The high-lift locks and dams along the Columbia and Snake Rivers in Washington and Oregon provide a contrast to the Ohio River emergency situation. These dams create relatively large deep reservoirs that are used to produce hydropower. Free flow through a lock at one of these projects does not constitute a major portion of the total river flow and the loss of reservoir storage results primarily in a loss of power production.

10.5.7 Debris Control

Material that drifts along waterways includes sediment, damaged barges, timber, ice floes, and the like. Chamber siting and guide and guard wall design (see EM 1110-2-1611) influence the extent to which waterway debris tends to enter the upper approach. These materials are of concern to navigation; valve, gate, and flow passage operation; and general maintenance of chamber and approaches. Primary hydraulic concerns are:

a. flow patterns and operational procedures directed toward flushing surface (floating) material over the upper sill, through the lock chamber, and out of the lower approach;
b. trash bars and trashracks at culvert intakes designed for exclusion of submerged materials from the filling-and-emptying system;
c. selection and design of the gates (see EM 1110-2-2703) and sills for reliable operation in the presence of both surface and submerged debris and for maintenance removal of unusual materials; and
d. identification of locations along the flow passage boundaries and the chamber floor at which long-term accumulations, physical damage, and other major inspection and maintenance concerns exist.

10.6 FILLING-AND-EMPTYING FEATURE DESIGN

10.6.1 General

The following paragraphs identify preliminary calculations required for very-low-lift (SG and SPF), low-lift (SP, BL1, and BL2), and high-lift (HB4 and B8) designs. See Table 10-4 for design type definitions. For lifts near design-type limits, ranges 5 to 10 ft and 30 to 40 ft economic cost/capacity studies may require the review of both a lower-lift design (normally with lower initial cost) and a higher-lift design (normally with greater capacity).

10.6.2 Sill Spacing Parameters

Preliminary layouts required for navigation, geotechnical, and structural studies require the sill spacing to be estimated early in the design process. Since the usable length is fully committed to navigation, the actual chamber length is usable length plus the gate length plus a safety clearance value.

10.6.2.1 Lower and Upper Miter Gates. The lower miter gate swing requires about 60 ft for 110 ft clear width locks and, similarly, 46 ft for 84 ft widths. Design practice is to provide a spacing of about 10 ft to accommodate obstructions and clearance at the upper sill and clearance at the lower leaf while the leaf is approaching the fully recessed position. Typical dimensions are listed in Table 10-5.

10.6.2.2 Lower and Upper Sector Gates. Requirements are similar to miter gate installations. For example, Leland Bowman Lock, which has a clear width of 110 ft and usable length of 1,200 ft has a 1,270 ft spacing between sector gate pintles. Large tows and small vessels near sector gates (Figure 10-9) require secure moorings and slow gate operation in order to prevent drift. Usable length based on clearance, as in Section 10.6.2.1, is therefore greater than a usable length based on chamber conditions.

10.6.2.3 Lower Miter and Upper Submergible Tainter Gates. The tainter gate trunnion is located and recessed within the chamber at Lower Granite Lock.

TABLE 10-5. Miter Gate Dimensions (ft)

Clear Width	110	110	84
Usable Length	1,200	600	600
Leaf Extension	60	60	46
Clearance	10	10	9
Pintle-to-Pintle	1,270	670	655

Clearance factors at the lower pool are the same as found in Section 10.6.2.1; protection for the tainter gate is an additional concern at higher pool levels. Typical dimensions in feet are:

1. clear width = 86
2. usable length = 675
3. lower leaf extension = 52
4. lower miter pintle to tainter gate trunnion = 728
5. lower miter pintle to sill face = 749 (varies)
6. clearance at lower pool = 22.

Spillway tainter gate structural details are suggested as appropriate for tainter gates on lock sills (EM 1110-2-2703).

10.6.2.4 Other Gates. Navigation inconvenience at lower pool (rising single-leaf vertical lift gates) and clearance for opening at upper pool (submergible or rising single or double-leaf vertical lift gates) preclude a significant reduction in sill spacing by using narrower gates. Gate designs are detailed in EM 1110-2-2703.

10.6.3 Sill Spacing

For preliminary layouts, sill spacing is based on usable length and miter gate or sector gate leaf extension; approximately 10 ft is added to provide a combined sill and gate clearance. Final gate selection considers structural, mechanical, and economic factors in addition to hydraulics and may result in an alternate gate and a small change in sill location.

10.6.4 Location of Intake Structures

The chamber inflow hydrograph (flow rate Q as a function of time t) is finalized during hydraulic feature design; however, estimates of flow are required before these details are known. Intake structures are located so that lockage flows are a minimum liability to navigation and also satisfy other site-specific constraints. Navigation conditions are often determined by means of hydraulics models (see EM 1110-2-1611) which require preliminary estimates of lock inflow rate.

10.6.5 Lock Filling

Computer programs are available to provide Q as a function of t for the lift and geometry of the new lock. Should operation time be greater than desired, then system size (culverts) is increased; additional costs as compared to the existing lock are anticipated. Should operation time be less than desired, then system size may be decreased. Idealized hydrography as shown in Figure 10-16, may also be used to establish preliminary estimates of lock inflow. The volume of inflow, using a discharge Q as a function of time t, is set equal to the change in lock chamber water volume. The following guidelines identify rapid filling times (small T values) for existing designs.

10.6.5.1 Very-Low-Lift Designs. For SG locks, the gate opening rate and pattern are adjusted in the prototype to accommodate various lift, vessel, and approach conditions. For SPF locks, valve pattern and port openings are adjusted in the prototype for the same reasons. Operation times near 10 min are the minimum achievable for acceptable chamber performance. For small SG chambers with recreational traffic, lower lifts, and adequate submergence, an operation time nearer 5 min may be appropriate.

10.6.5.2 Low-Lift Designs. For SP locks, acceptable chamber performance is obtained during hydraulic feature design for a specific filling time and specific commercial traffic (9 ft draft tows) because of tested relationships

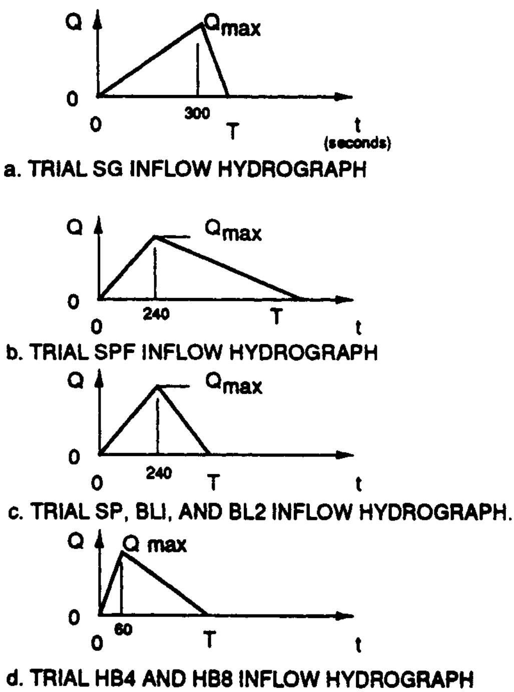


FIGURE 10-16. Idealized Lock Filling Hydrographs for Preliminary Estimates of Lock Inflow.

among lift, chamber dimensions, submergence, port dimensions, baffles, and valving. An 8 min operation time is a common goal for lifts near midrange, 25 ft. Predesign estimates of SP operation time for an 84 by 600 ft chamber and 4 min valving are shown in Figure 10-17. Neither BL2 nor BL1 designs have as comprehensive a set of operation time versus submergence data as do side-port systems. For these systems, a filling time of 8 min and a valve time of 4 min are suggested for preliminary inflow estimates for the entire low-lift range.

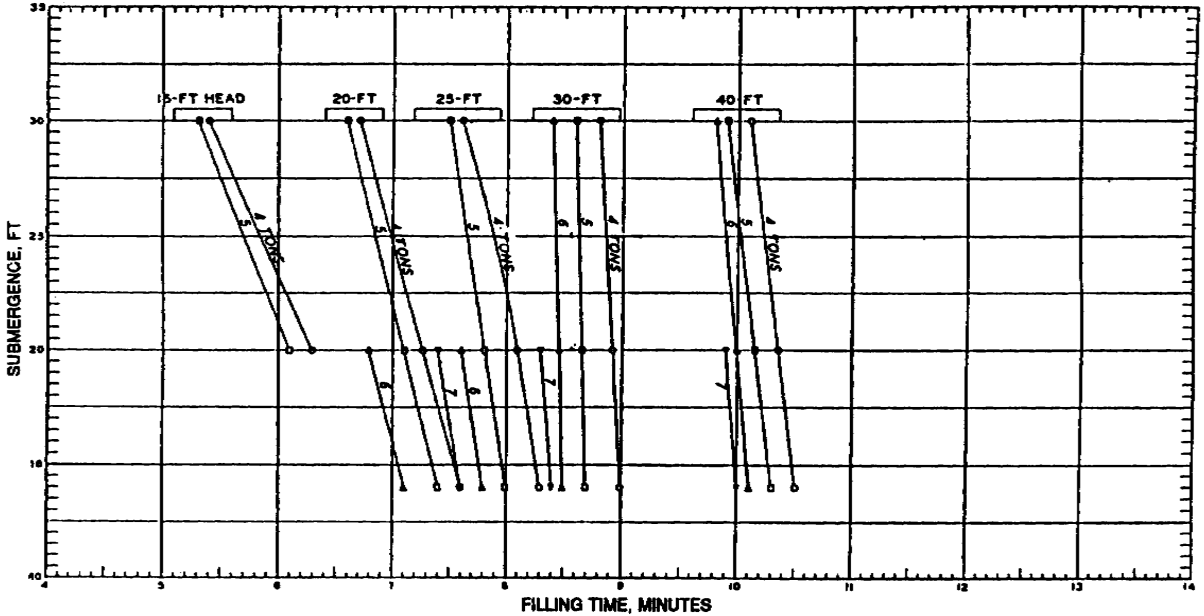
10.6.5.3 High-Lift Designs. HB4 and HB8 chamber details are variable during design only with extensive laboratory testing regarding chamber performance. Both systems are designed for rapid valving (1 min) and rapid filling. Prototype filling times for these systems range from 8 to 12 min for lifts from 40 to 100 ft.

10.6.6 Chamber Depth

Chamber depth *Dc* (Figure 10-18) for design purposes is the depth of water in the lock during navigation lockage conditions. The minimum depth corresponds to the minimum tailwater elevation and the maximum depth to the maximum upper pool elevation for which lockage is planned. The choice of the chamber floor elevation must include safety and economic considerations. The time of entry and the filling/emptying time are decreased and the cost of the structure is increased as the chamber depth is increased. Safety is improved as the chamber depth is increased. The minimum chamber depth must have a filling time that is slow enough not to violate the 5 ton hawser stress guidance. It may be that the sill depth requirements will limit the minimum chamber depth. An economic analysis using the incremental delays in lock transits for increments of tailwater/headwater durations versus the incremental structural cost of providing various chamber depths can be employed to optimize the benefit-to-cost ratio. Submergence is defined as the difference in elevation between lower pool and chamber floor. Cushion is defined as the elevation difference between vessel keel and chamber floor for zero velocity conditions.

10.6.6.1 Very-Low-Lift Designs (0–10 ft). These locks have been constructed with chamber floor at navigation channel bed elevation. The submergence has therefore been established by upstream and downstream channel conditions rather than chamber performance.

10.6.6.2 Low-Lift Designs (10–30/40 ft). The minimum submergence for optimum filling/emptying time for side-port locks is the tow draft plus one half the side-port spacing. For a 9 ft draft tow in a 110 ft wide lock, the optimum minimum submergence is 14 + 9 = 23 ft. When excavation costs asso-



NOTE: HAWSER STRESSES WERE MEASURED ON [illegible]-BARGE TOW (11,110 TONS DISPLACEMENT) POSITIONED 36 FT FROM UPSTREAM MITER GATE PINTLES.

SUBMERGENCE IS THE DIFFERENCE IN ELEVATION BETWEEN LOWER POOL AND THE LOCK CHAMBER FLOOR.

PORTS WERE 2 FT WIDE BY 3 FT HIGH AT THE THROAT.

PERMISSIBLE FILLING TIMES TO KEEP HAWSER STRESSES WITHIN 4-, 5-, 6-, AND 7-TON LIMITS 84′ × 600′ CHAMBER

FIGURE 10-17. Filling Time Test Data. Side-Port Data Are From Model Tests, the Prototype Will Operate About 10% Faster.

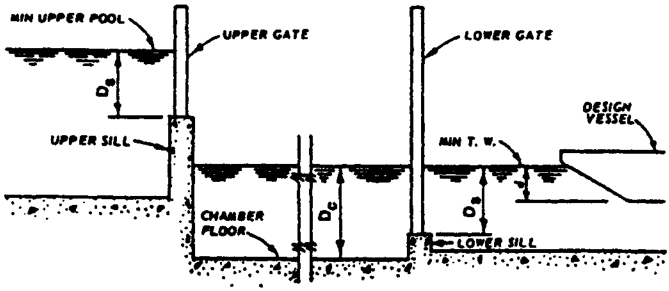


FIGURE 10-18. Sill Elevations.

ciated with deep submergence are significant, then the lateral BL2 system has been used. Using 16 ft submergence plus 7 ft lateral culvert total height = 23 ft as the criterion, then for lifts less than about 25 ft, BL2 is not an economical alternative to SP systems. For lifts above 25 ft the BL2 design has been used instead of the SP design provided reduced excavation represents a major economic factor as compared to the expense of lateral culverts and risk during single or nonsynchronous culvert operation being operationally acceptable. The high-lift HB4 type of design is expected to be an effective alternative to BL2 designs, although use in 1,200 ft chambers has yet to be studied. The auxiliary lock, BL1, is normally set so that submergence is equal to that of the main lock.

10.6.6.3 High-Lift Designs (30/40–100 ft). Submergence values of 15 ft have been used for Bay Springs and Lower Granite Locks. The extreme excavation measured from lower pool to the lowest invert in the crossover area is 34 ft for HB4 design and 41 ft for HB8 design. The HB8 design with modified crossover culverts has been model-tested for a 69.5 ft lift, 14 ft draft tows, 5 ft cushion, and 86 by 675 ft chamber with no evidence of unsatisfactory performance. The VB4 designs, which have similar manifolds but modified crossovers as compared to HB4, have been model-tested for lifts ranging from 30 to 100 ft, for a range of lifts and chamber sizes; prototype experience is available with these designs. The HB4 design (modified) was considered for a 130 ft lift, 84 by 600 ft chamber; however, the Coosa River locks project was terminated for economic rather than operational reasons.

10.6.7 Sill Elevation

Sill depth D_S (Figure 10-18) for design purposes is the depth of water over the sill during navigation lockage conditions. The minimum depth corresponds to the minimum tailwater elevation for the lower sill and to the minimum upper pool elevation for the upper sill.

10.6.8 Sill Elevation Guidance

The choice of sill depth must include safety and economic considerations. As the sill depth is either the same or less than the chamber depth, it becomes the governing factor for safety and tow entrance time. A sill depth less than 1.5 times the tow draft (1.5*d*), except for very-low-lift (0–10 ft) locks, should not be considered due to safety reasons. A normal entrance speed of approximately 3 mph requires a sill depth of 2*d* to avoid excessive squat and loss of vessel speed control. When gate operating clearance above the floor to allow for some accumulation of trash is necessary, either a 2 or 3 ft height of sill above the floor or a floor recess is provided. Since there is very little difference in the cost of the sill versus the cost of the gate, the sill elevation should be kept as low as possible for ease of tow entry and exit and for safety reasons due to the possibility of grounding caused by squat and/or ice accumulation on the barge hull. The upper sill depth should be equal to or greater than the lower sill depth. Consideration can be given to a much greater depth if a need to pass emergency traffic during a loss of pool situation or other exigency is projected.

10.6.9 Location of Outlet Structures

Constraints are so that lockage flows (emptying) are a minimum liability to navigation and satisfy other site-specific concerns and so that satisfactory chamber performance is retained. For sector gates the outflow point is lower gates, and discharge is directly into the lower approach channel. For culvert systems the outflow is either into the approach channel (by means of bottom or side manifolds) or, when possible, into the main river remote from the approach, or by a division of flow between main river and approach canal. Three specific preliminary information needs are as follows.

10.6.9.1 Navigation. Discharge hydrographs are required for studies (EM 1110-2-1611) of navigability in the lower approach. Control during emptying is at the outlet ports which, in design, can be modified to increase peak flows (decrease operation time). For preliminary calculation the outflow hydrograph is made identical to the inflow hydrograph (Figure 10-16) although a 10 to 20% decrease for peak flow during emptying is not uncommon.

10.6.9.2 Channel Stability. Discharge hydrographs are required; the estimates (Section 10.6.9.1) are used for preliminary studies of bed and bank stability. Structures for energy dissipation and stone for bed and bank protection are often required.

10.6.9.3 Stages. For remote outlets, the differential between stage at the outlet location and stage in the lower approach channel affects lower gate oper-

TABLE 10-6. Outlet Structure Type

Project (Typical)	Outlet Structure Type
Vermillion	Sector gate
Lock 52	Channel side; one multiported structure
Ozark	Remote; one with two ports
Belleville Main	Remote; two with one port
Willow Is. Aux.	Remote; one structure with one port
Bay Springs	Channel bed; two multiported structures
Lower Granite	Remote; one structure with two ports

ation. Values are required for the navigable range of hydrologic conditions at the project.

10.6.10 Typical Outlet Locations

The outlet structure types in Table 10-6 are from Figures 10-9 to 10-15.

10.7 VERY-LOW-LIFT DESIGNS

10.7.1 General

Relatively small static and dynamic hydraulic loadings occur for locks with very low lifts (water-surface differential $H < 10$ ft). In addition, constraints with regard to chamber performance (filling time and hawser stress) are normally sufficiently flexible so that adjustments to the field operating procedure, rather than design information, are used to optimize chamber performance. These adjustments are as follows.

10.7.1.1 Sector Gate (SG) Locks. To obtain satisfactory chamber performance, the gate opening rate, pattern, and duration are finalized in the prototype.

10.7.1.2 Side-Port-and-Flume (SPF) Locks. The number and sizing of open ports are chosen during prototype operation.

10.7.2 Sector Gate Design Concept

The gate and recess, shown in Figure 10-9, are geometrically formed so that the minimum dimension between recess lip and recess boundary equals the clear opening at the lock center line. Flow is distributed across the width of the chamber since the recesses in addition to the center-line opening, are flow passages.

10.7.3 Hydraulic Evaluation

Sector gate lock studies include the following fundamental evaluations.

10.7.3.1 Operation Time. Longer filling-and-emptying times are expected for projects requiring larger chamber water-surface areas or having higher lifts. The size and shape of the flow passages through the gate recesses affect the rate of flow into and out of the chamber as well as affecting the mooring conditions immediately downstream from the gate. The primary means of altering the operation time for a specific sector gate design is by optimizing the rate and extent of gate opening. The values in Table 10-7 apply to constant rate gate opening tests for the Sacramento Barge Canal Lock.

10.7.3.2 Chamber Mooring Conditions. Velocities and turbulence near the upper gate during filling and the lower gate during emptying are unfavorable as mooring conditions. For example, a usable chamber length of about 540 ft rather than 640 ft based on gate location is suggested for the Sacramento Barge Canal Lock. An alternate solution is slow gate operation.

10.7.3.3 Hydraulic Loadings. The forces required to open and close the sector gate under normal and reverse flows are sensitive to gate lip shape. Loadings are presented in EM 1110-2-2703.

TABLE 10-7. Constant Rate Gate Opening Tests (Sacramento Barge Canal)

			Filling	Emptying	Maximum Gate Opening	
Stage (ft)	Lift (ft)	Gate Opening Rate (deg/min)	Time T (min)	Time T (min)	Filling (deg)	Emptying (deg)
34.5	21	0.33	13.7	20.1	4.6	6.7
		0.66	9.4	13.7	6.2	9.0
29.5	12	0.33	12.5	15.1	4.1	5.0
		0.66	8.8	10.7	5.8	7.1
		1.00	7.2	8.8	7.2	8.8
22.5	6	0.33	12.6	14.3	4.2	4.7
		0.66	8.1	10.1	5.4	6.7
		1.00	7.2	7.8	7.2	7.8

Stage is referenced to upper gate sill.

10.7.3.4 Flow Rate. The chamber water-surface elevation is evaluated by simultaneously numerically integrating flow rate Q and elevation z relationships:

$$Q = cb_g h^{3/2} \tag{10-1}$$

$$Q = A_L \frac{dz}{dt} \tag{10-2}$$

where

- c = Coefficient that is assumed constant for free flow conditions, but under submerged conditions gradually decreases with increased submergence (see Figure 10-19)
- b_g = Effective gate opening that includes the center-line opening and the gaps through the recesses
- h = Upper pool water-surface height above the upper sill
- z = Chamber water-surface height above the upper sill
- A_L = Lock chamber water-surface area.
- dz/dt = Rate of change of the chamber water-surface elevation

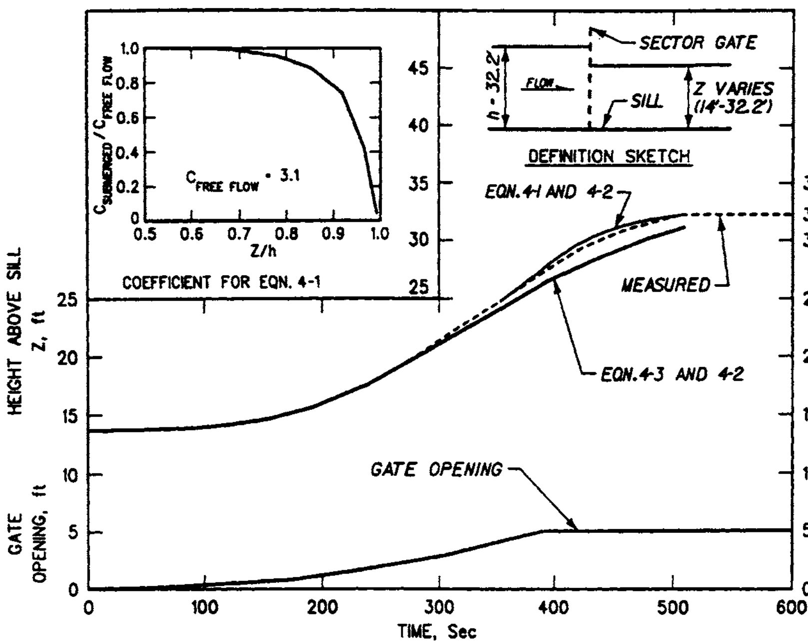


FIGURE 10-19. Example of Sector Gate Filling (Algiers Lock).

Filling is initiated with the upper gates closed and the lock chamber at lower pool level. An example of a calculation for Algiers Lock is shown in Figure 10-19. For filling with continuously submerged flow ($z/h > 0.7$), Equation 10-2 in conjunction with the orifice equation is probably more reliable than the foregoing procedure. The flow rate is expressed as

$$Q = cb_g h(2g(h-z))^{1/2} \tag{10-3}$$

in which the coefficient c is about 0.55. Model and prototype experience, with provision for field adjustment of the sector gate opening pattern, is an essential part of the hydraulic design of sector gate locks.

10.7.4 Side-Port Flume Designs

Prototype study data are available from the US Army Engineer District, Louisville. These data include valve operation schedules and operation times for lifts experienced at Locks 52 and 53 (temporary locks). Qualitative information regarding port sizing, flume and chamber performance, and operational experience are also available. These locks have not been model-tested, so generalized design data are not available.

10.8 CULVERT-TO-CHAMBER SYSTEMS

10.8.1 General

The arrangement and sizing of the chamber ports affect chamber performance (hawser stresses, for example) as well as operation time. The flow through the culvert-to-chamber system is bidirectional; that is, the ports are discharge orifices during filling and intakes during emptying. These requirements have resulted in a small set of effective designs that are suited to a reasonably broad range of design constraints.

10.8.2 Chamber Port Arrangements

The layout of lateral (BL1 and BL2) design is based on model tests conducted for Greenup and Markland Locks. Small variations in locating and sizing the lateral manifolds have been adopted for design and have performed acceptably in the field. The location of the SP manifolds relative to chamber length follows specific guidelines outlined in Section 10.14. The location of the longitudinal manifolds (HB4 and HB8) is invariant; that is, all chamber details are required to be identical to Bay Springs Lock, B4, or Lower Granite Lock, B8.

10.8.3 Flow Passage Areas

The discharge orifice areas (chamber ports for filling and outlet ports for emptying) are primary elements for meeting operation time criteria. The most rapid systems are ones in which these areas are maximized while energy losses within the culverts and manifolds (and valving times) are minimized. Flow passage areas for five lock designs are listed in Table 10-8.

10.8.3.1 Filling. Systems that contract from main culvert to chamber (HB8 at Lower Granite) adapt to requirements for rapid filling by using relatively large culverts with minimum losses attributable to culvert features. Energy dissipation is primarily by baffling within the chamber. Systems that expand from main culvert to chamber (BL1, BL2, HB4) adapt to requirements for rapid filling by using relatively large ports with significant energy dissipa-

TABLE 10-8. Flow Passage Areas

		Description (Size = Width × Height, ft × ft)					
Location	Item	Ozark	Willow Island	Belleville	Willow Island	Bay Springs	Lower Granite
Chamber Ports	Type	SP	BL2	BL2	BL1	VB4	VB8
	Number,[a] N_1	14	24	18	18	24	12
	Size (Face)[b]	3.25 × 3.50	3.69 × 4.70	1.83 × 2.08	1.83 × 2.08	1.5 × 3.5	1.25 × 3.46
	Size (Throat)	2.54 × 3.50	2.75 × 4.07	NA[c]	NA	NA	NA
Chamber Manifolds	Number,[d] N_2	1	1	9	8	2	4
	Shape	Box	Box	Stepped	Stepped	Box	Box
	Size (Maximum)	12 × 12	16 × 18	8 × 5	8 × 5	14 × 9	14 × 9
	Size	12 × 12	16 × 18	15 × 16	14 × 16	14 × 14	12 × 22
Outlet Ports	Number,[d] N_3	1	1	1	1	16	1
	Shape	Basin	Basin	Basin	Basin	Stepped	Basin
	Size	17 × 12	20 × 16	19 × 16	20 × 16	3 × 6	12 × 14
Operation	Evaluation	Area Ratios					
Filling	2 + 1	0.78	0.65	NA	NA	NA	NA
	$3 + N_1 \times 1$; $3 + N_1 \times 2$	0.90; 1.16	0.69; 1.07	0.58	0.58	1.00	1.16
Filling	$4 + N_2 \times 3$	1.00	1.00	0.67	0.70	0.78	1.10
	$4 + N_1 \times N_2 \times 1$	0.90	0.69	0.39	0.41	0.78	1.27
Emptying	$N_1 \times N_2 \times 1 + N_3 \times 5$	0.78	1.30	2.04	1.72	0.88	1.24
	$4 + N_3 \times 5$	0.71	0.90	0.79	0.70	0.68	1.57

[a]Per manifold.
[b]Excludes 0.5 to 1.5 ft radius surface contour.
[c]Not applicable.
[d]Per culvert.

tion occurring within the culverts as well as within the manifold sections. For example, in the Bailey Lock prototype BL2 design (16 ports per lateral, 8 laterals per culvert) the loss is about 3 times greater than for a streamlined system. Similarly, for the Greenup system (18 ports per lateral, 11 laterals per culvert) the loss is nearly 6 times greater.

10.8.3.2 Emptying. Chamber ports are inefficient as intakes. Efficient systems that contract from chamber to outlet (VB8 at Lower Granite) are designed for longer emptying than filling times and for energy dissipation concentrated downstream from the outlet. Expanding systems (SP at Ozark and VB4 at Bay Springs) tend toward more rapid emptying, although relatively greater losses are caused by chamber ports and manifolds. Deep submergence for water-surface elevations near the upper pool reduces the possibility of cavitation within the chamber ports and manifolds during emptying.

10.8.4 Chamber Ports, Baffles, and Manifolds

Ports for SP systems are discussed in Section 10.14. Port and manifold geometries, as used in BL1 and BL2 systems, are shown in Figure 10-20. For lateral systems, ports within a manifold are equally spaced on each wall and equally sized (2.08 ft high by 1.83 ft wide is common); the number of ports per manifold and the number of manifolds vary among designs. The manifold roof is horizontal, whereas the interior sidewalls are stepped as shown in Figure 10-20. Port extensions are used when flow alignment, particularly from the upstream ports, during filling is of concern. Baffling is provided at adjacent manifold walls by offsetting ports between manifolds. Ports are chamfered with regard to outflow (filling) and inflow (emptying). Ports for high-lift designs (HB4 and HB8) experience high velocities and are chamfered for flow in either direction as shown in Figure 10-20. Tee-baffle walls and baffles located on lock and culvert walls are required. The ratios of total port area to manifold areas are 1.000 and 0.865 for HB4 and HB8, respectively. These values near unity, similar to SP systems, are required for efficiency for bidirectional operation. Values substantially greater, 1.7 for the Greenup system shown in Figure 10-20, are efficient with regard to emptying (i.e., as an intake) but are relatively inefficient for filling.

10.9 OUTLET SYSTEMS

10.9.1 General

Discharge outlet systems are the orifice controls for the emptying operation. The dominant chamber performance constraint is the operation time

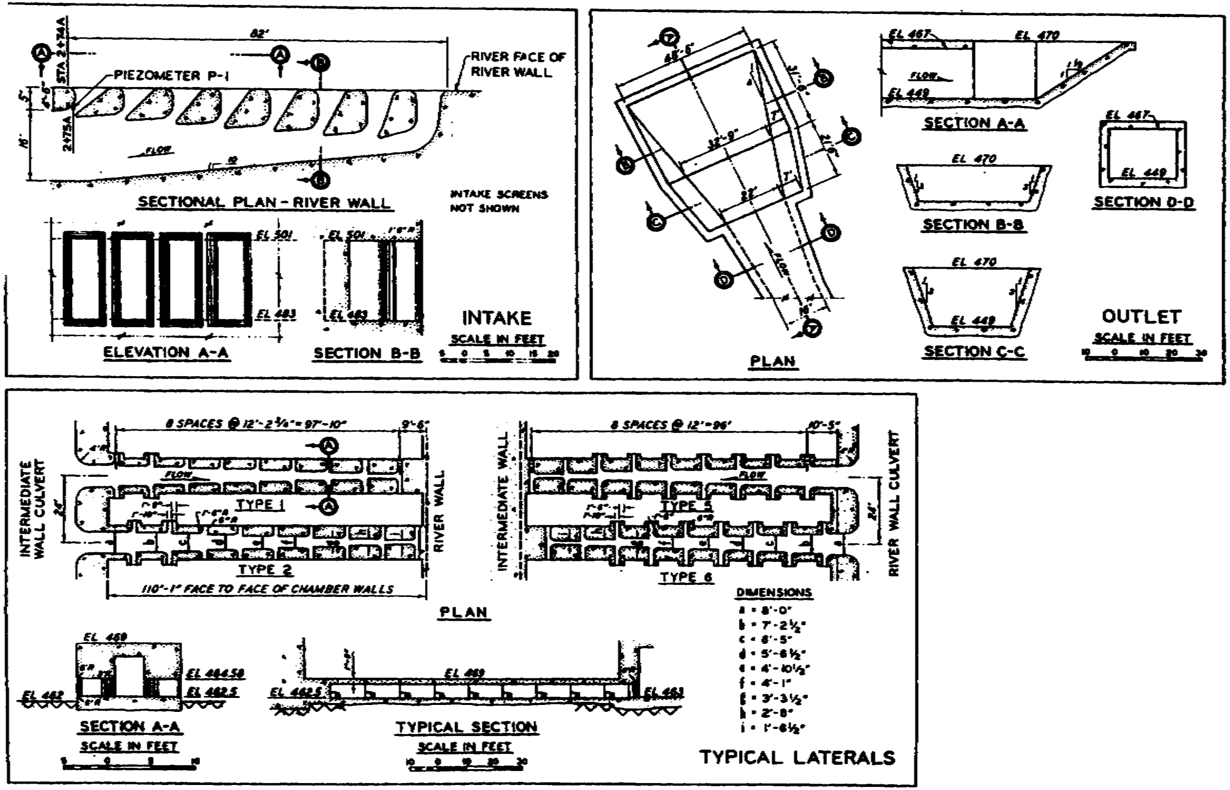


FIGURE 10-20. Hydraulic System Details.

as affected by outlet sizing. The dominant downstream approach channel constraint is the navigation facility as affected by discharge hydrographs and outlet location. The following distinctions regard sizing.

10.9.1.1 Expanding Systems. The outlet port area is made greater than the chamber port area normally for the purpose of decreasing operation time. Concurrently, greater energy losses occur within the system (i.e., the chamber ports are not efficient as intakes) so that outflow velocities are also decreased. Both effects are favorable for low-lift locks. For high-lift locks, low local pressures and high pressure fluctuations are associated with expanding high-velocity systems.

10.9.1.2 Contracting Systems. The outlet port area is made equal to or less than the chamber port area. The common purposes are to raise the hydraulic grade line within the system and to reduce discharge rates within the approach channel at the expense of increased operation time. Contracting systems are best suited for high-lift designs and are rarely appropriate for low lifts.

10.9.2 Design Types

Outlet design variations occur because of options regarding location. General types are outlined in Figure 10-21 as follows.

10.9.2.1 Manifolds in Approach Channel Floor. One or several manifolds from each emptying culvert extend across the approach channel. The Bay Springs design results in uniform transverse flow distribution near the lock. The new Bonneville design requires the channel expansion (as tested for the Dalles lock) to be initiated near the manifolds in order to attain a uniform flow within the approach channel. The new Bonneville system contracts (discharge port area to chamber port area ratio equals 0.83) whereas the Bay Springs system expands (ratio equals 1.14). The St. Anthony Falls Lower Lock is an example of large expansion and uses four lateral manifolds branching from one discharge culvert

10.9.2.2 Manifolds in Guide and Guard Wall. Two such expanding systems are shown in Figure 10-21. The Trinity River model-test manifold discharges directly into the lock approach. The New Cumberland Main Lock discharge is subdivided by the main lock into river, main approach, and auxiliary approach components. The Trinity River system requires baffles at each port. These types of approach-channel manifolds are low cost and are well suited for low-lift projects when higher velocities and turbulence in the approach near the lock are acceptable (as contrasted with remote outlets, Sections 10.9.2.3 and 10.9.2.4).

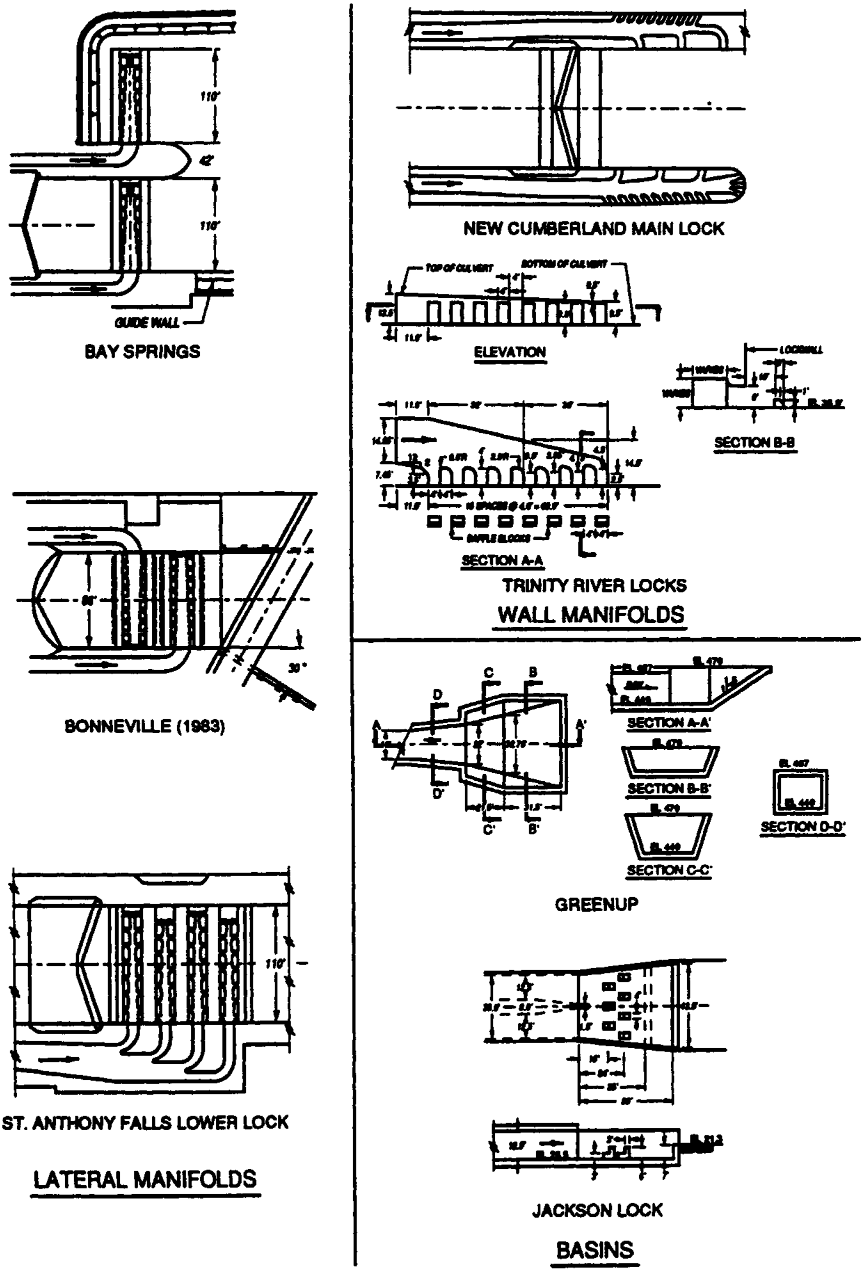


FIGURE 10-21. Outlet Types.

10.9.2.3 Basins. Normally, and when economically feasible, the most favorable outlet location with regard to navigation is in the main river remote from the lock approach. Basins used for these outlets are as shown in Figure 10-21. The Greenup Lock type basin is relatively deeply submerged so that energy dissipation within the flow exterior to the basin is acceptable. The Jackson Lock type is designed as a stilling basin; test data pertain to designs without and with various spacings of baffle blocks and end sill. Lower Granite (high-lift) uses a Greenup-type basin with a contraction (discharge port area to chamber port area ratio equals 0.80). Ozark Lock (low-lift) uses a Jackson Lock unbaffled basin with an expansion (ratio equals 1.29).

10.9.2.4 Other Types. The outlet may be placed (usually remotely) so that other outlet structures as used elsewhere (outlet works, for example) suit a site-specific design. The structure must:

1. provide conditions (particularly with regard to navigation) in the lower approach that are satisfactory;
2. have expansion or contraction conditions between chamber manifolds and outlet that are acceptable with regard to chamber performance; and
3. provide a capability for reliably handling structural and hydraulic needs (particularly large intermittent discharges) during lock chamber emptying.

10.10 INTAKES

10.10.1 General

Intake flows are essentially unidirectional. The design pertains to filling only and seeks to accomplish the following objectives.

10.10.1.1 Navigation and Sedimentation. The location and orientation are such that adverse effects on navigation and channel sedimentation are avoided.

10.10.1.2 Debris and Ice. The elimination of debris from the culvert normally requires trashracks at the intakes. These are placed on the wall face (common) or immediately within the wall structure (Lower Granite). The reduction of clogging at the intakes and sediment transport into the culverts is of obvious benefit in terms of lock maintenance. Trashracks must be secured for small reverse loadings that occur during lock chamber overfill.

10.10.1.3 Velocities. The intake is designed as a highly convergent streamlined manifold having the concurrent objectives of equal flow distribution through the ports and small energy loss. Small energy loss contributes to efficient lock filling and, for two-culvert systems, enables equal culvert flows to be attained with substantially different intake configurations. Low velocities through the trashbars place less stress (and reduce the possibility of flow-induced vibration) on the exposed structural elements. Existing rack structures are generally conservative for peak velocities less than 4 feet per second (fps); higher velocities may require special attention (EM 1110-2-1602; EM 1110-2-2602).

10.10.1.4 Vorticity. The formation of large vortices at lock intakes is considered highly undesirable because of hazard to small vessels, imbalance between culvert flows, and damage to trashrack. The elimination of vortex action for a specific filling pattern requires studies of the following items.

10.10.1.4.1 Local Geometry and Flow Constraints. Geologic and structural features, such as the shape and orientation of guide and guard walls, may introduce vorticity into the intake flow. Similarly, adjacent spillway or river flows may result in vortex formation under a particular format of overall project operation. An intake located outside the approach channel so that navigation is not affected by vorticity over the intake structure is advantageous at many projects.

10.10.1.4.2 Structure Type. Generally, for small submergence, intakes are long and shallow with numerous ports (8 to 12 are not uncommon); a uniform distribution of flows over the length of the structure tends to reduce vortex formation. Short and high intakes (four ports at Lower Granite) may function satisfactorily when deeply submerged.

10.10.1.4.3 Submergence. Deeply submerged intakes (see EM 1110-2-1602) are genially less prone to vorticity than those with shallow submergence. Extrapolating submergence effects based solely on changing upper pool levels as compared to changing intake elevation (with fixed pool level) is questionable because of local geometry.

10.10.1.4.4 Operation. Vorticity intensifies as the valve is opened and persists during and sometimes beyond the Iock-filling period. Operational situations, particularly valve opening times and maximum flow values, are important.

10.10.2 Design Types

Examples of intake structures are shown in Figure 10-22 with layout parameters listed in Table 10-9. These and other intakes have been studied (physical hydraulic models) and adopted for site-specific application.

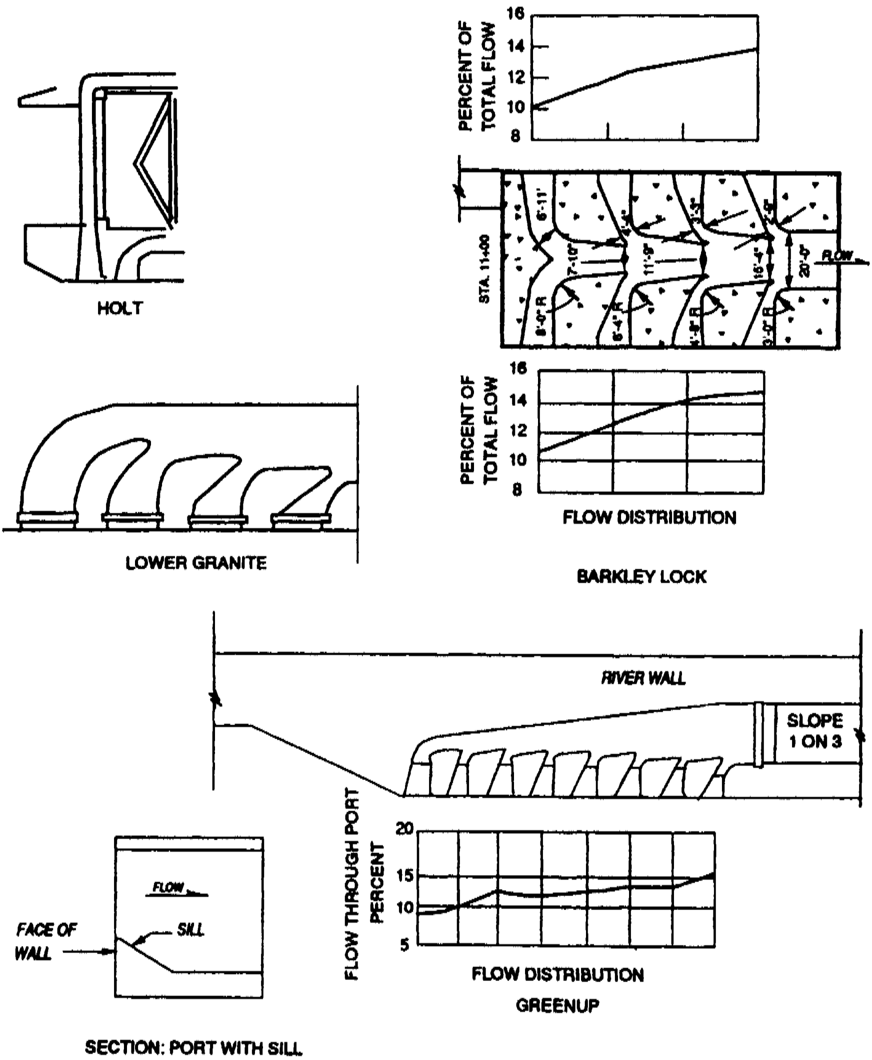


FIGURE 10-22. Intake Structures.

TABLE 10-9. Examples of Model-Tested Intake Layouts

Lock	Lift (ft)	Q (cfs)	No. of Ports	Port Height (ft)	Port Width (ft)	Manifold Length (ft)	Pier Thickness (ft)	Submergence (ft)
Holt	63.6	7,000	1	31	18	18	NA	46.5
Lower Granite	105.0	13,600	4	30	8	47	5	58.0
Greenup	32	7,000	8	12[b]	8	99	5	14.0
Bay Springs	84	9,100	10	14	7	115	5	48.0
Dardanelle	54	6,000	13	13	7	151	5	24.0
Barkley	57	4,400	2 × 4	13	7.5	66	12	29.0
Dardanelle	54	6,000	2 × 7	13	7	79	5	24.0

[a]Dimensions exclude rounding at the wall face.
[b]4-ft-high sill, culvert at intake 18 ft wide by 16 ft high.

10.11 FILLING-AND-EMPTYING VALVE SYSTEMS

10.11.1 General

Recent lock designs use reverse tainter valves for flow control. Alternate valve types provide less desirable hydraulic, structural, operational, or economic conditions. The normal tainter valve (skinplate upstream) has been replaced for lock design by the reverse tainter valve (skinplate downstream) because of the ease of regulating air demand for the latter design. The normal valve is not precluded from lock design (particularly as an emptying valve); however, current practice is to use the reverse tainter valve for emptying as well as filling. Comprehensive design guidance presented in EM 1110-2-1610 provides details regarding valve types, loadings, losses, and the like; this discussion is limited to an overview of the valves as they relate to the overall filling-and-emptying arrangement. The following sections deal exclusively with reverse tainter valves.

10.11.2 Valve Sizing

By using streamlined contractions upstream and gradual expansions downstream, the valves can be sized substantially smaller than the main culvert section. Section area changes are commonly accomplished by a change in culvert roof elevation rather than offsetting the culvert walls. Large valves (e.g., 18 ft high by 16 ft wide) are designed for the new Gallipolis low-lift lock. The extreme contraction-and-expansion design is at the Lower Granite high-lift lock, which, for a 22 ft high by 12 ft wide main culvert, uses 14 ft high by 12 ft wide filling-and emptying valves. The advantage of small valves is lower cost particularly, because of the greater loading, at high-lift projects. Higher velocities and lower pressures at the valve location occur for small valve designs during valve full open conditions.

10.11.3 Valve Siting

Structural, operational, and economic considerations for valve siting must satisfy the following hydraulics topics.

10.11.3.1 Position Along the Culvert. The filling valve, downstream from the intake manifold, and the emptying valve, upstream from the outlet, are separated from the culvert-to-chamber system by a streamlined transition conduit. The fundamental requirement is that the distribution of flow into and out of the culvert-to-chamber system is not unbalanced due to nonuniformity in the adjacent main conduit flow. Current guidance requires a distance of 6.5 culvert heights (as measured at the filling valve) between the filling valve and the culvert-to chamber system (EM 1110-2-1610).

10.11.3.2 Elevation. The hydraulic consideration is pressure downstream from the valves that contributes to air entrainment and cavitation. Entrained air, particularly for low-lift locks, may accumulate in the culverts as a pressurized air mass with the potential for bursting through the water surface and through vents and wells. Well-mixed air is more common for high velocities associated with high-lift locks and, when excessive, causes a frothy condition at the outflow water surface. Guidance on air entrainment is included in EM 1110-2-1610. Cavitation, particularly at high-lift locks, may cause surface damage to culvert walls, valve seals, and other exposed valve components. A condition in which cavitation causes pressure shock waves to occur in the flow downstream from the valve is resolved during design by either air venting the low-pressure region below the valve so that air rather than vapor pockets occur; setting the valve at a low elevation so that vapor pressures do not occur; or using a less efficient system also so that vapor pressures do not occur. Guidance for avoiding cavitation is included in EM 1110-2-1610.

10.12 CULVERT LAYOUTS

10.12.1 General

The culvert geometry includes bends, contractions, expansions, junctions, bifurcations, and the like, as required to resolve the plan and profile layout of the intake, valves, culvert-to-chamber, and outlet systems. Recent designs use rectangular culverts. The aspect ratios (height to width) near 1.0 are common although values as extreme as 1.6 and 0.6 have occasionally been used. Ratios at the valve location (18:16, 14:12, 12:12, etc.) are always near unity for valve structure and economy reasons. Hydraulic design parameters, such as those included in EM 1110-2-1602, are equally applicable to lock culverts provided allowance is made for the normally short spacing between components and the unsteady nature of lock flows.

10.12.2 Contracting and Expanding Systems

System sizing (intake, filling valve, culvert-to-chamber, emptying valve, and outlet) establishes the extent of section area and shape changes within the culvert. These changes are particularly susceptible to separation at boundaries introducing energy loss, turbulence, and, particularly for high-lift locks, cavitation effects into the flow. To avoid these problems, expansions are normally gradual (roof expansions 1*V*:6*H* to 1*V*:10*H* are common) and contractions are streamlined. The flare of each SP port sidewall, for example, is about 3 degrees for filling; rounding at port intakes and outlets has ranged from about 0.5 to 2.0 ft.

10.13 OTHER HYDRAULIC DESIGN FEATURES

10.13.1 Surge Reduction

Surge reduction is accomplished by:

a. slower filling-and-emptying systems or longer valving. This results in lower surges at the expense of long operation time;
b. surge basins to suppress the rapid drawdown (filling) or upwelling (emptying) during the normally brief period of rapid change in discharge rate;
c. hydraulic surge control methods as a means of removing or adding water to a small canal located between two locks. Additional volume is needed during filling of the lower lock; removal is needed during emptying upstream;
d. staged lifts to reduce peak flow rates (as in (a)) at substantial increase in operation time; and
e. broad approach channels to lower surges; that is, canalized systems are more susceptible to surge effects than are broad river systems.

10.13.2 Computational Aids

Surge reduction is discussed in EM 1110-2-1606. Surge height calculations as presented in EM 1110-2-1606 are computer accessible in the US Army Corps of Engineers CORPS program library. For long canals or more complex geometries, study aids such as more comprehensive analytical (computer-based) solutions or physical model studies are needed.

10.13.3 Impact Barrier

The purpose of a barrier is to provide an energy-absorbing device for barge tows to prevent damage to the gates in the event of a collision. Four such devices have been considered for use to protect lower miter gates. They are wire rope fenders, steel collision barriers, concrete collision barriers, and rope system impact barriers. The rope system impact barrier has been studied for use upstream of upper miter gates (the other three types appear less suitable for upstream use). These barriers are discussed in EM 1110-2-2602.

10.13.4 Water Saving

During periods of low water on canalized waterways, a sufficient supply of water is required to maintain navigation pools at or above planned normal pool elevations. These factors affect pool elevation:

a. available hydrologic water supply;
b. leakage, seepage, and multipurpose (hydroelectric plant, for example) consumption;
c. water requirements for lockages;
d. pumpage or diversion, and return flow (where applicable); and
e. evaporation.

The water supply must be equal to or exceed the algebraic sum of the other factors in order to maintain the navigation pools. The water supply may consist of the natural flow of the stream, the supply furnished by storage reservoirs, or a combination of the two. A thorough investigation should be made for an item when any doubt exists as to the adequacy of the water supply.

10.13.5 Dewatering

Hydraulic concerns during dewatering include bulkhead locations, pumping facilities, and outflow conditions. Dewatering exerts an extreme static loading on structural elements and requires specific considerations during lock structural detail design (see EM 1110-2-2703 and EM 1110-2-2602). Structures used for emergency closure are normally suitable for dewatering.

10.13.6 Emergency Closure (General Emergency Situations)

Emergency situations occur at navigation locks when a lock gate becomes inoperative in an open or partially open position while a head differential exists between the chamber and upper or lower pool. Although the cause may be mechanical failure, the more frequent cause is a navigation error that holds the gate partially open. Although no universally accepted definition of emergency closure exists, the required action is generally understood to be that a closure structure must be rapidly placed in flowing water under head differential.

10.13.7 Consequences of Pool Loss

The main consequences of upper pool loss downstream of the project are due to the flood wave. Hazardous navigation conditions and rapid flooding of riverfront property are extreme possibilities. A less severe flood wave will commonly interfere with the operation of private and commercial boat docks. Upstream impacts of pool loss include the following.

a. Economic and safety problems occur at commercial and recreational boat terminals. Long periods of navigation suspension have a severe adverse impact on the economy of an entire region. The primary loss on major navigating systems is loss of navigation channel.
b. In many areas, small riverfront communities depend on the maintenance of normal pool for water supply. Loss of pool during low-flow periods causes inconvenience and, possibly, health and fire hazards.
c. Rapid loss of pool and resulting drawdown causes bank instability. This problem is especially severe where important structures, highways, or railroads are located in the reach of instability.
d. A navigation project that includes hydropower loses some or all of its power-generating capability in the case of upper pool loss.
e. Upstream pool loss causes a severe and adverse impact on fish and wildlife.
f. Upstream pool loss affects other site-specific factors particularly during extremely low upper pools.

10.13.8 Types of Emergency Closure Systems

A broad range of structures are in place as emergency closure devices at existing US locks. Operational and economic considerations, rather than purely functional, limit the choices for new designs. Structural details are available in EM 1110-2-2703 and in other references. Examples of the more common closure devices are as follows.

10.13.8.1 Bulkheads. The most common type of emergency closure for locks and spillway gate bays is a bulkhead consisting of one or more sections and commonly constructed of welded high-strength steel. A watertight skin plate is generally provided on the upstream side. Top and bottom seals, side seals, and roller assemblies complete the structure. The roller assemblies bear on bearing plates constructed in pier or lock wall recesses. The vertical height of the structure may vary from 3 to 12 ft depending on design constraints of a specific project. Several individual units are usually required for complete lock or dam closure.

Most designs do not permit water flowing over and under the bulkhead units during lowering. Stacking units may be required for successful placement. Some bulkheads are equipped with an overflow plate attached to the top truss. The purpose of such design is to utilize bulkheads for flushing ice and debris when necessary. If bulkheads are designed for placement in flowing water, hydraulic model studies of previously untested situations are needed.

The units are either stored at the locks or retained in dogged position over the dam. In the former case, an overhead gantry crane is used to transport the individual units to the lock. The first unit is dogged over the bay or the lock and the next unit is moved from storage, latched to the first one, and then the assembly is lowered and dogged a second time. Additional bulkhead units are latched to the assembly until closure is achieved.

Another method of placement uses a stiff-leg derrick positioned at the lock. The derrick raises and places individual units in bulkhead recesses. Additional units are added until closure is achieved. During lowering, the assembly is held in place by a stop log carriage.

10.13.8.2 Vertical Lift Gates. Emergency lift gates are either the single-leaf or the double-leaf type (see EM 1110-2-2703). The cost of the gate, storage arrangements, and hoist mechanisms for either type vary according to river stage and project (closure) lift. Economic studies are ultimately used to choose between single- or double-leaf gates. Double-leaf vertical lift gates have been constructed at several navigation locks on the Ohio River navigation system; other navigation systems use single-leaf vertical lift gates. In either system the gates are stored in submerged position under the lock emergency sill upstream of the upper miter gates. The double-leaf construction permits the utilization of locks as floodways when the river stage prohibits navigation. An emergency-closure single-leaf gate is illustrated in Figure 10-23.

For the double-leaf type design used in the Ohio River navigation system, only the downstream leaf is designed to permit closure in flowing water. However, the vertical height of one leaf is sufficient to effect closure under unbalanced head (flowing water) up to normal pool level. Should closure be required for stages above normal pool, then both leaves can be raised, since upstream and downstream heads are balanced. The operation of double-leaf-type emergency closure is shown in EM 1110-2-2703. For the single-leaf emergency gate, provisions must be made in the design to allow closure.

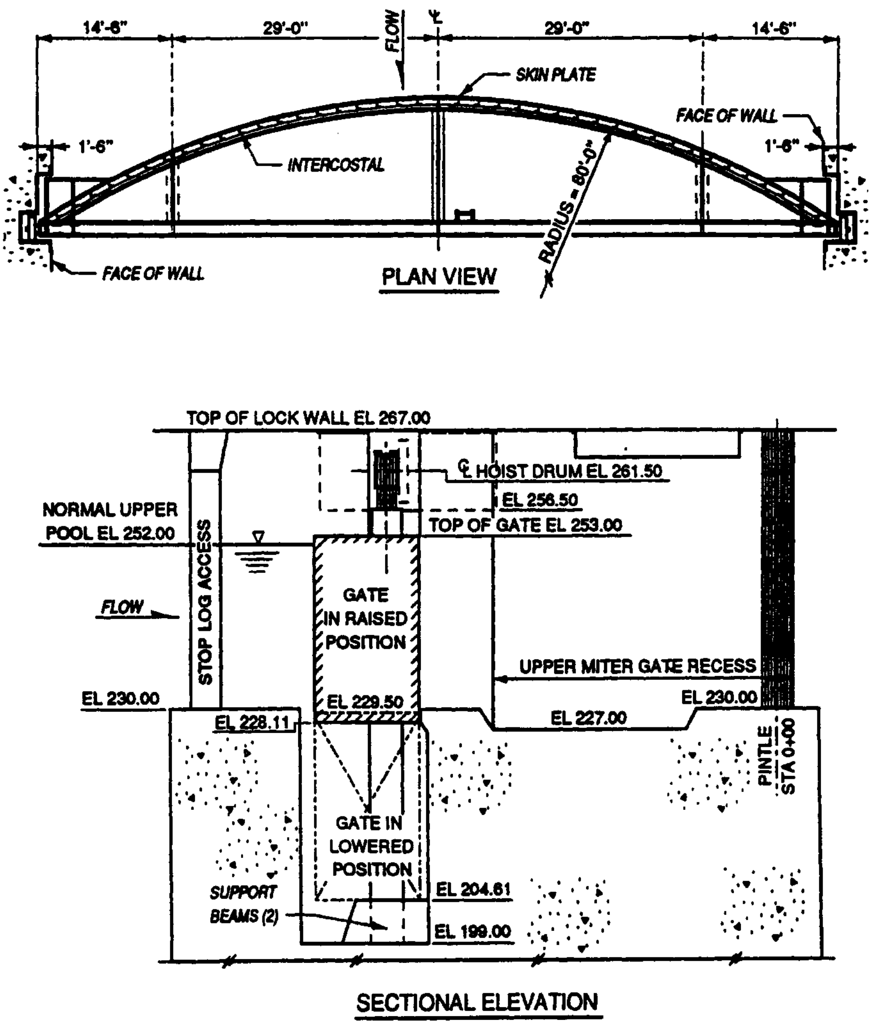


FIGURE 10-23. Emergency Closure System, Vertical Lift Gate

10.13.8.3 Upstream Emergency Dam. A type of emergency closure designed and constructed by the US Army Engineer District, Nashville, for several locks on the Cumberland River navigation system is an emergency dam. This consists of several wickets that remain submerged on the floor of the emergency sill during normal locking operation, but are raised into position during emergency conditions. Each wicket is raised individually by means of a chain hoist, sheaves, and a winch located on the top of the lock wall. When wicket No. 1 is in the lowered position, the landward hoist chain fits into a recess in the lock wall. As the first wicket is raised, it also raises the attached hoist chain of the next wicket. After locking the first wicket in position, the sheave is passed over to the riverward side and the second wicket is raised, which also raises the hoist chain for the third wicket. The operation continues in this manner until all wickets are raised. Similar closures have been constructed and operated on other navigation systems. In the original design, the wickets were constructed with a flat skin plate; however, hydraulic model testing includes a curved skin plate.

10.13.8.4 Other Systems. Stop logs, commonly consisting of wooden beams, can be placed in recesses upstream of spillway gates or lock miter gates using a hoisting mechanism. However, in general, operating heads on the dam usually must be reduced before placement. Since this arrangement would result in partial or total loss of pool, they cannot be considered a true emergency closure. Bulkheads, described in Section 10.13.8.1, are sometimes designated as stop logs. An older type of emergency closure is used for the auxiliary lock at McAlpine Lock and Dam on the Ohio River system. This type of closure includes a separate horizontal beam placed across the top of the lock walls with a derrick. Closure panels are vertically placed between the beam and the concrete sill to complete the closure operation.

Submergible tainter gates are another alternative for emergency closure. Under normal operating conditions, the gates rest in a recess built in the emergency sill, upstream of the upper miter gates. During emergency closure, the gates are lifted to position by cables. Provisions must be made to clean the gate recess periodically to free it of accumulated silt and debris.

10.13.9 Design Loadings

An overview of design loadings (EM 1110-2-2703) is as follows. Hydrodynamic forces result from the water flowing under the emergency closures. On emergency bulkheads, these forces can result in hydraulic uplift or downpull depending on the design. In order to lower bulkheads in flowing water, the uplift force must be less than the submerged weight of the bulkhead. Knowledge of the magnitude of hydraulic downpull is important for the design of the hoisting machinery. Overflow and underflow on emergency bulkheads are undesirable from the standpoint of hydrodynamic forces and should not be used. Hydraulic model studies are sometimes required to determine forces for a particular design.

The weight of the bulkhead is to be determined in the usual manner considering the structural elements and members of the closure. The majority of the bulkheads are of structural steel, but aluminum bulkheads have been used. The submerged weight is important in considering the ability to lower the closure structure in flowing water.

Frictional forces develop along the side support of closure structures. The magnitude of these forces depends on the type of bearings and side seals as well as on other loadings (e.g., as in the preceding example). Reference is made to EM 1110-2-2703 for details.

Some types of emergency closure systems, notably vertical lift gates, can be used in a dual role serving also as lock gates. Barge impact loads are considered for these designs. Reference is made to EM 1110-2-2703 for the magnitude of such loads.

Ice forces are considered, depending on the climatic condition at the location of the closure.

10.14 EXAMPLE DESIGN OF SIDE-PORT SYSTEMS

10.14.1 Description

A typical sidewall port filling-and-emptying system has a longitudinal culvert in each lock wall extending from the upper pool to the lower pool, with a streamlined intake at the upstream end and a diffusion device at the downstream end. Flow is distributed into and out of the lock chamber by short ports between the longitudinal culverts and the sides of the lock chamber. Two valves are required in each longitudinal culvert, one between the intake and the manifold of lock chamber ports to fill the lock and the other between the manifold of lock chamber ports and the discharge diffuser to release flow in the emptying operation. This discussion is concerned with design of that portion of the system between the filling-and-emptying valves.

10.14.2 Port Size

From data collected in model tests of an 84 ft wide lock, three 110 ft wide locks, and a 150 ft wide lock, desirable cross-sectional area for a port is plotted against lock width in Figure 10-24. Studies have shown that the extent of the primary zone of diffusion of a submerged jet is a function of jet size and thus the optimum size port is dependent only on lock chamber width. Certainly the degree of surface turbulence in the lock chamber increases as the lift increases and/or as the submergence (difference in elevation between initial lower pool and the lock chamber floor) decreases, but distribution of turbulence across the chamber is independent of lift and submergence. For the 655 ft long by 84 ft wide Jonesvine Lock, a 6.0 sq ft port resulted in good distribution of turbulence and ports of other sizes were not tested. In the model study of the 670 ft long by 110 ft wide Arkansas River low-lift locks, ports with cross-sectional areas of 6.0, 8.9, 10.4, and 12.7 sq ft were tested. The 6.0 sq ft ports definitely were too small as the jets from the ports were diffused prior to reaching the opposite side of the lock chamber. This resulted in boils with excess turbulence along the center of the lock chamber and caused large hawser forces on a moored tow. Conditions produced by the 8.9 and 10.4 sq ft

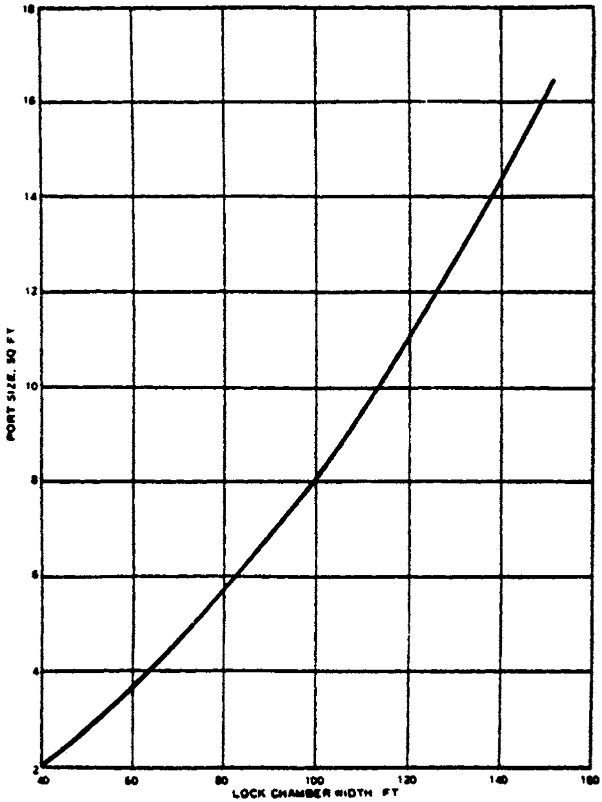


FIGURE 10-24. Recommended Port Size.

ports were rated as satisfactory. With the 12.7 sq ft ports longer filling times were required for acceptable hawser forces than with either the 8.9 or 10.4 sq ft ports. Also, turbulence was considered excessive and it was concluded that this port was too large. In model tests for the 1,270 ft long by 110 ft wide Cannelton Lock, ports 8.4 and 11.2 sq ft in cross-sectional area were observed. For equal filling times more favorable hawser forces resulted with the 11.2 sq ft ports. Upon completion of the tests for the Cannelton Lock and the Arkansas River low-lift locks, engineers involved in both studies agreed that the 8.9 and 10.4 sq ft ports tested in the Arkansas model resulted in more favorable turbulence conditions across the lock chamber than did the 11.2 sq ft port tested for Cannelton Lock. These engineers are of the opinion that a slightly better design for the filling system for Cannelton Lock could have been developed if a port 9.0 to 10.0 sq ft in cross-sectional area had been used. For the 1,265 ft long by 110 ft wide New Cumberland Lock, a port 9.5 sq ft in cross-sectional area was selected. In the model of the 1,290 ft long by 150 ft wide Mississippi River–Gulf Outlet Ship Lock, a port 16.2 sq ft in cross-sectional area results in good distribution of turbulence across the lock chamber. Ports of other sizes have not been tested. Obviously a variation in port size of about 5% to either side of that recommended is acceptable.

10.14.3 Port Spacing

Ports in one wall should be staggered with respect to the ports in the other wall so that the jets issuing from one culvert will pass between jets from the other culvert. If ports are spaced too close together, the jets from the opposite walls will meet and boils will form near the center of the lock, resulting in large hawser forces. If spacing between the ports is too great, the port jets will tend to stray, resulting in some areas of essentially no turbulence and other areas of excess turbulence.

Again the areas of excess turbulence will cause large hawser forces. Recommended spacing for the ports in a lock wall is given in Figure 10-25. In a

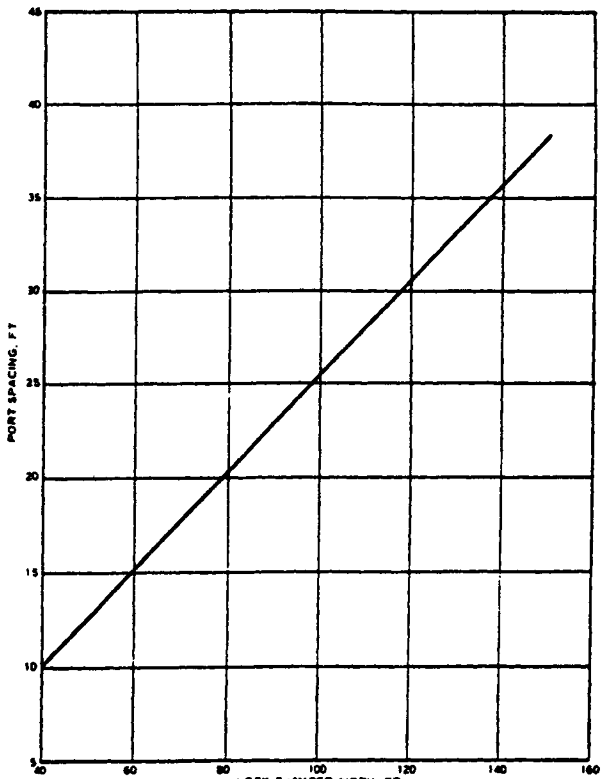


FIGURE 10-25. Recommended Port Spacing.

110 ft wide lock, a spacing of 28 ft center to center for the ports in each wall has been found to be optimum in several model studies. For locks of other widths there are few significant data. In an 84 ft wide lock, spacings of 22 and 20 ft were tested, and the 20 ft spacing was preferred although a 21.5 ft spacing is indicated in Figure 10-25. In a 150 ft wide lock, only a spacing of 38 ft has been observed and this appears to give satisfactory conditions. Certainly spacing is not so critical that variation of 1 ft on either side of that recommended would result in a noticeable change in conditions.

10.14.4 Number of Ports

Following selection of port size and spacing, the next consideration is the number of ports that is feasible for the particular lock. In this connection the port group must be centered with respect to the length of the lock chamber, and it must extend over at least 50% of the lock chamber length. If the port group does not extend over at least 50% of the chamber length, hawser forces on a tow in either the upstream or downstream half of the lock chamber will be greater than those on a tow that occupies the entire lock chamber. The greater the extent of the port group the better, but usually structural considerations will limit the port group to about 60% of the lock chamber length.

10.14.5 Culvert Size

After the number of ports that can be accommodated is fixed, then the desirable size for the culverts in the lock walls can be determined. In each culvert the ratio of the total cross-sectional area of the ports to the cross sectional area of culvert should be about 0.95. If the cross-sectional area of the ports is as large as or larger than the cross-sectional area of the culvert, poor distribution of flow from the port manifold will result to the extent that during peak discharge of a filling operation, flow is likely to be drawn from the lock chamber by the upstream ports. On the other hand, if the port-to-culvert area ratio is too small, filling time will be sacrificed without a noticeable improvement in conditions in the lock chamber.

10.14.6 Culvert Shape

A culvert square in cross-section allows for easy forming of the culvert and port and results in good hydraulic efficiency. However, forming advantages can be maintained with a rectangular cross-section and as long as the minimum dimension is at least two thirds of the maximum dimension there will be very little loss in hydraulic efficiency. Frequently wall stability and valve design are simplified by making the height of the culvert greater than the width.

10.14.7 Port Shape

There is an advantage in a rectangular port with the width equal to about two thirds of the height. With a narrow port there is less downstream component in the jet issuing from the port due to the velocity of the flow passing the port in the wall culvert. On the other hand, turbulence in the lock chamber is better distributed with a square port rather than with a long narrow port. Long narrow ports result in unstable jets with severe concentrations of turbulence. A port in which the width is about $^{2}/_{3}$ of the height is as narrow as is feasible without the risk of unstable jets. Also it has been found to be beneficial to flare the sides of the port by as much as, but never more than, 3 degrees. The length of a port should never be less than 3 times its width and a length of about 4 times the width is desirable. A port suitable for a 110 ft wide lock is shown in Figure 10-26.

10.14.8 Port Deflectors

Even with properly designed ports there is likely to be a downstream component in the jets issuing from the upstream ports in the manifold where velocity of flow past the ports is quite high. Triangular deflectors that tend to counteract this downstream component are beneficial at the upstream third of the ports in the manifold. These deflectors reduce the peak upstream hawser force and allow about a 5% decrease in permissible filling time. (Permissible filling or emptying time is the time required to fill or empty the lock without causing hawser forces on a rigidly moored tow in a model to exceed the equivalent of 5 tons prototype.) Unfortunately, general rules for the design of deflectors for various size ports have not been developed. Satisfactory conditions in a 110 ft wide lock were obtained with a deflector as shown in Figure 10-27. This deflector can be formed by a wall on the lock floor or by a recess in the lock floor. If a recess-type deflector is used, then recesses probably will be desirable at all ports. In this case triangular recesses are suggested for the half of the ports in the upstream end of the lock chamber and rectangular recesses for the ports in the downstream end of the chamber.

10.14.9 Angled Ports

There are data from tests in which ports were angled upstream in attempts to gain the same benefits as those gained with deflectors. In all cases conditions resulting with angled ports were not as favorable as those obtained with deflectors.

10.14.10 Required Submergence

Submergence is defined as the difference in elevation between the lower pool and the lock chamber floor. The greater the submergence, the faster is

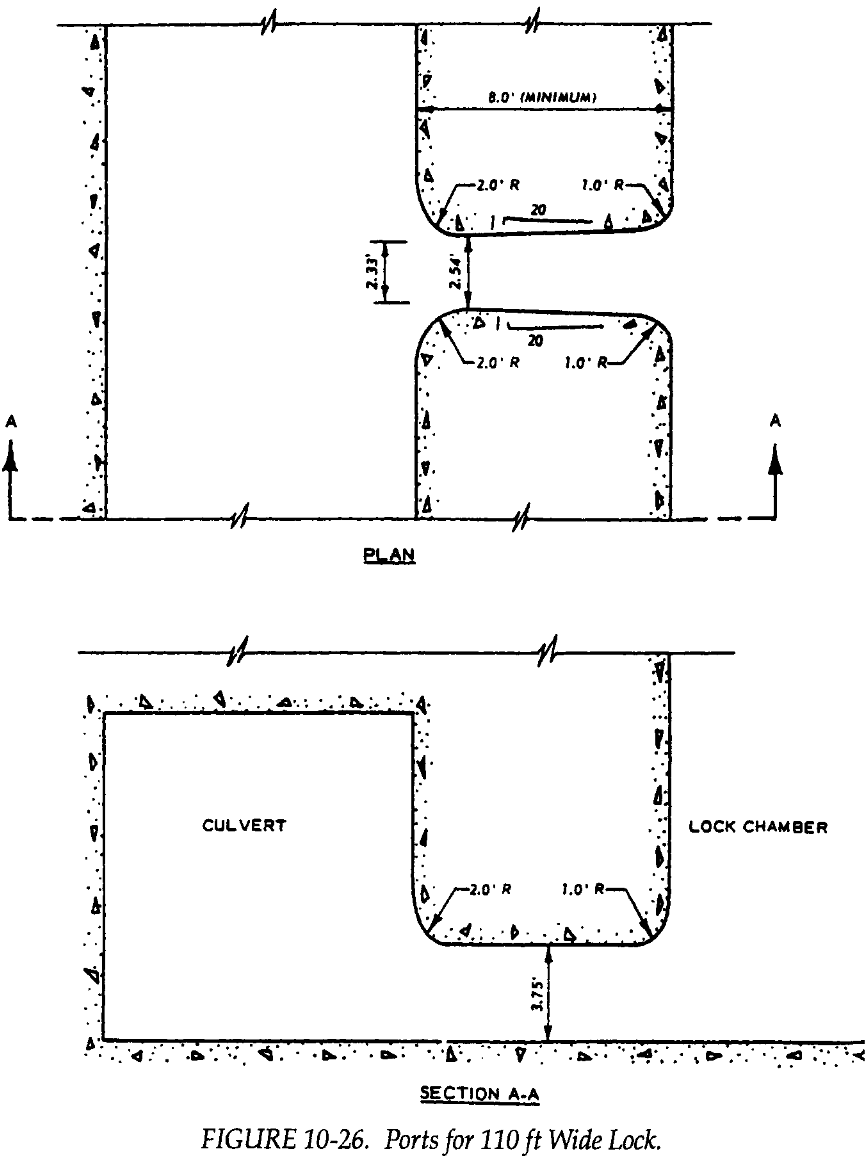


FIGURE 10-26. Ports for 110 ft Wide Lock.

the permissible filling time. However, in many cases each foot of submergence provided is quite costly and the designer needs to know the minimum submergence at which satisfactory operation can be expected. Data from various width locks indicate that the jets from the ports expand in an upward direction at the same rate as they expand horizontally. Thus a clear space between the bottom of the vessel using the lock and the floor of the lock chamber equal to half of the port spacing is required to prevent direct

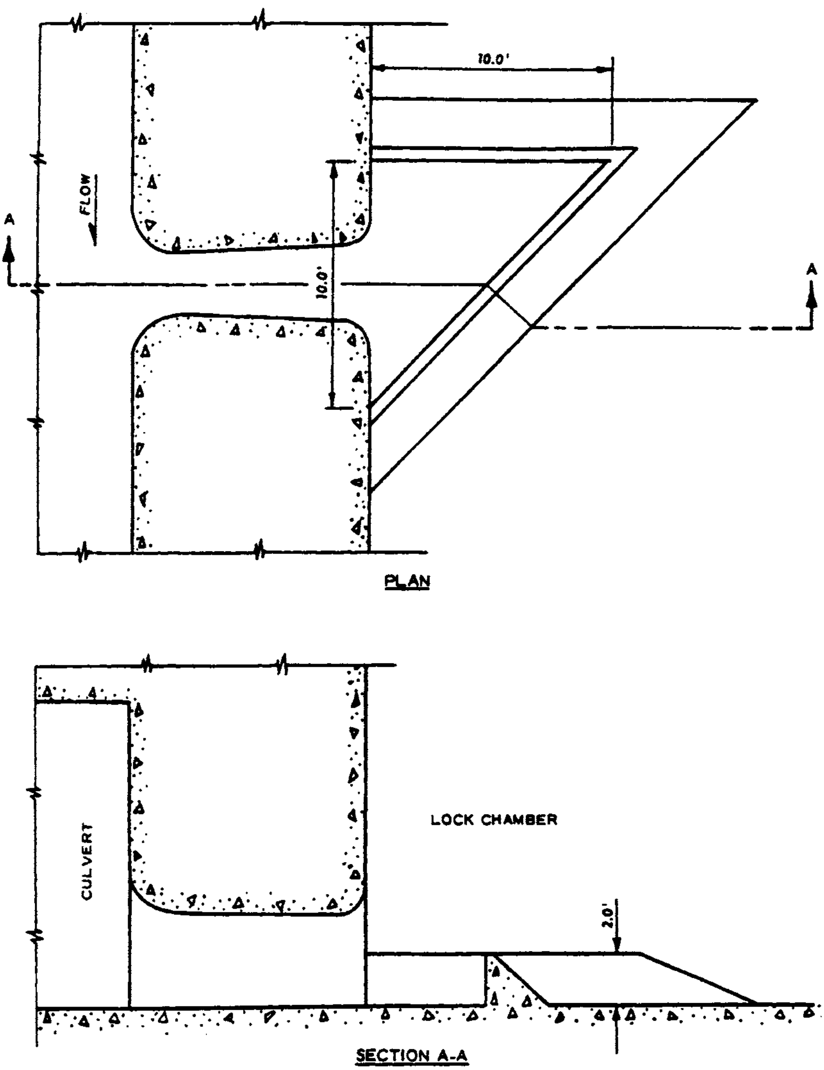


FIGURE 10-27. Port Deflector for 110 ft Wide Lock.

action of the port jets against the bottom of the vessel. In a 110 ft wide lock designed for tows of 9 ft draft, a submergence of 23 ft should be provided (9 ft, draft of tow, plus 14 ft clear under tow, half of 28 ft port spacing). If a greater submergence than that suggested is provided, then permissible filling times will be shorter, but an increase in clear space under the tow of 100% will allow a decrease in permissible filling time of only 10%. On the other hand, a decrease in the suggested clear space under the tow of only 20% will require a 20% increase in permissible filling time.

10.14.11 Ports Above Chamber Floor

It may be structurally desirable to have the ports enter the lock chamber at an elevation higher than the lock chamber floor. If this is the case then the ports should be angled down so that the jets are directed at the base of the opposite chamber wall such as was done at the Eisenhower and Snell Locks. Of course the ports should never enter the chamber at an elevation that will result in jets having a direct impact on a vessel using the lock.

10.14.12 Valve Position

During opening of the filling valves there are depressions in the pressure gradients in the culverts that extend from each valve to a section about 6.5 times the culvert height downstream from the valve. Thus it would be expected that there would be a deficiency in flow from ports placed in this zone. However, it is during the valve opening period that the discharge from the upstream ports is likely to be in excess of that desired. In a series of tests for Newburgh Lock it was found that, with the port manifold placed in positions that resulted in the first two and the first four ports being within the low pressure zone downstream from the valve, no differences in filling time or hawser forces could be detected from those obtained with the manifold placed so that all ports were outside the low pressure zone.

10.14.13 Culvert Transitions

If there are transitions in the culverts downstream from the filling valves or upstream from the emptying valves, all ports should be outside the transition zones as pressures in these zones will be modified even after the valves are fully open. Expansions downstream from the filling valves and contractions upstream from the emptying valves can be used to provide the optimum size culvert through the reach of the port manifold with smaller and thus less costly valves and bulkheads. Of course this will result in greater losses through the contracted reaches of the culvert and somewhat longer filling-and-emptying times.

10.14.14 Suggested Designs

A good design for a 670 by 110 ft lock would have 15 ports, as shown in Figure 10-26, from each of two 150 sq ft culverts (minimum dimension at least $^2/_3$ of maximum dimension) and deflectors as shown in Figure 10-27 on the 5 upstream ports in each culvert. If designed for tows of 9 ft draft, minimum lower pool would be 23 ft above the lock chamber floor.

Similarly a 1,270 by 110 ft lock would have 28 ports, as shown in Figure 10-26, from each of two 280 sq ft culverts, with deflectors as shown in Figure

10-27 on the 9 upstream ports in each culvert. Again for tows of 9 ft draft, minimum lower pool would be 23 ft above the lock chamber floor.

A 655 by 84 ft lock would require 18 ports, each with a throat area of 6 sq ft, from each of tow 115 sq ft culverts. Deflectors similar to that shown in Figure 10-27 would be installed on the 6 upstream ports in each culvert. If designed for tows of 9 ft draft, minimum lower pool would be 19.5 ft above the lock chamber floor.

10.14.15 Valve Times, Filling

In Figure 10-28 are plotted permissible filling times (hawser forces not in excess of the prototype equivalent of 5 tons in 1:25 scale models) for the designs described in Section 10.14.12. In Figure 10-29 are plotted the valve times required in the models for the permissible filling times shown in Figure 10-14. Also in Figure 10-29 are recommended valve times for use in prototype operation. Note that these valve times are essentially the same as were required in the models. It has been established from experience that a prototype lock will fill about 9% faster than will its 1:25 scale model but that condition in the prototype will be satisfactory if the valves are operated at a rate no faster than was required to limit hawser forces to 5 tons in the model. Thus filling times in the prototype will be about 9% faster than those shown in Figure 10-28.

Valve times required in the model for the 84 by 655 ft lock are not shown in Figure 10-29 because the culverts used in the tests for the Jonesville Lock were 15% smaller than are considered optimum. Actually with the smaller culverts a valve time of about 2 min was satisfactory for all lifts, but for optimum size culverts the valve times recommended in Figure 10-29 are considered more appropriate. These valve times were interpolated on the basis of the lock chamber length-to-width ratio. The greater the length-to-width ratio of the lock chamber, the greater are the permissible filling times and valve times. For other length-to-width ratios valve times should be interpolated from those shown.

10.14.16 Valve Times, Emptying

For emptying, allowable valve times vary with the length-to-width ratio of the lock chamber, as in filling; but unlike in filling, allowable valve times are relatively independent of lift. In a 670 by 110 ft lock a valve time of 2 min is satisfactory for all lifts. A 1,270 by 110 ft lock requires a 4 min valve time for all lifts.

10.14.17 Filling and Emptying Computations

The usual formula for computing lock filling and emptying is

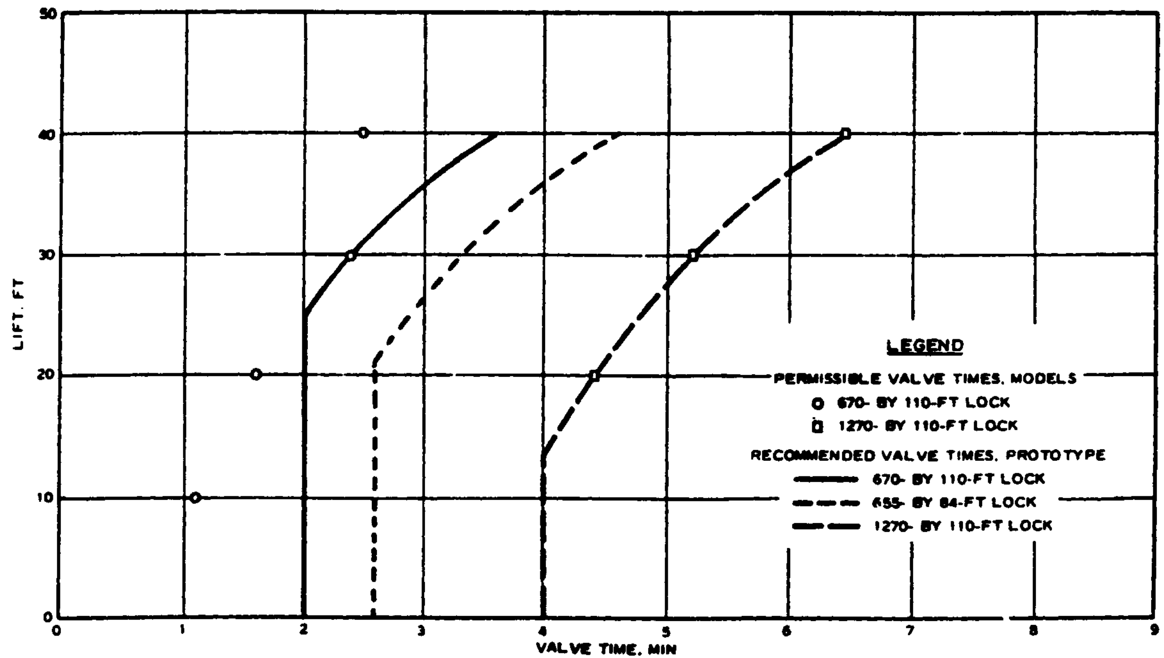


FIGURE 10-28. Permissible Filling Times—Model.

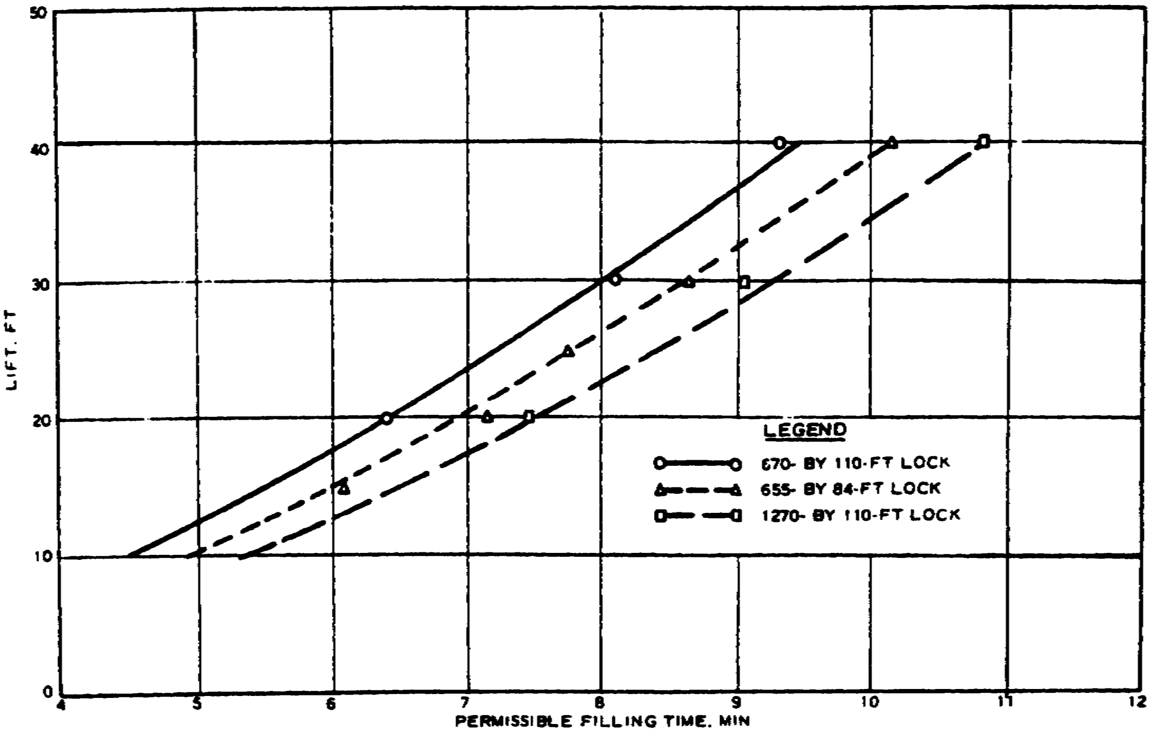


FIGURE 10-29. Valve Times—Filling.

$$T_v^{-Kt_v} = \frac{2A_L(\sqrt{H+d_f} - \sqrt{d_f})}{2C_L A_c \sqrt{2g}}$$

where

T = filling or emptying time, sec
K = a constant (value depends upon the valve opening pattern usually about 0.50)
t_v = valve opening time, sec
A_L = area of lock chamber, sq ft
H = lift, ft
d_f, d_e = overfill or overempty, ft
C_L = a coefficient (value depends upon losses in system)
A_C = cross-sectional area of culvert, sq ft
g = acceleration of gravity, ft per sec^2

For the systems described in Section 10.14.12 with intake and outlet structures essentially as shown in Figure 10-30, values for d and C_L are listed in Table 10-10. These values of C_L are 9% greater than those determined in 1:25 scale models

The total head loss through a filling-and-emptying system (H) is related to C_L thus:

$$H = \frac{1}{(C_L)^2} \times \frac{v^2}{2g}$$

where

v = velocity in wall culverts through the full open valve, ft per sec

For the systems described in Section 10.14.12, the total head loss is distributed as listed in Table 10-11.

10.14.18 Discussion

The sidewall port filling-and-emptying system is an excellent system for low-lift locks. Although data are given herein for lifts as great as 40 ft, general use of the system for lifts of more than about 30 ft is not recommended. Improper operation or malfunction of the valves will create conditions that are undesirable at low lifts but become dangerous at lifts of more than about 30 ft.

Compared with the bottom longitudinal filling-and-emptying system, which is used for high-lift locks, the sidewall port system has favorable dis-

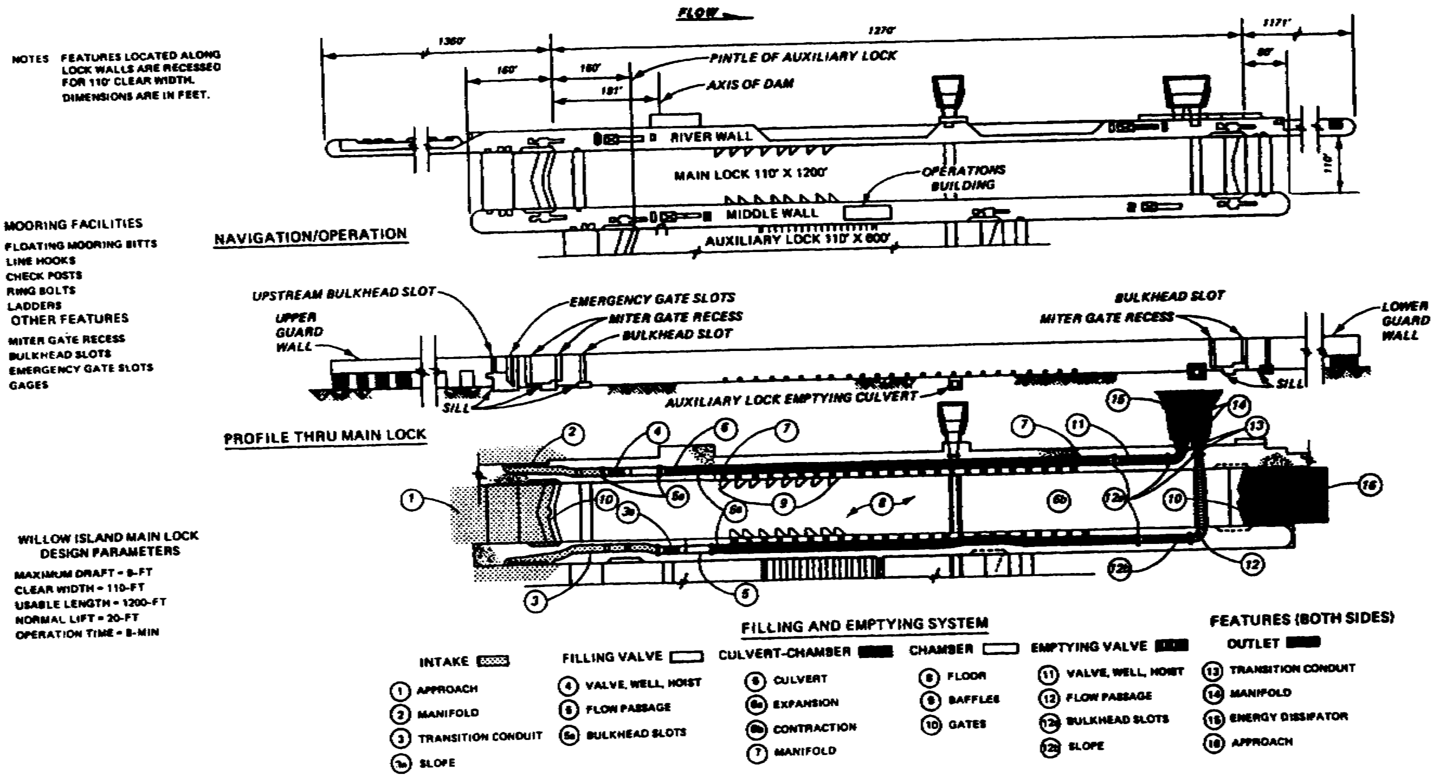


FIGURE 10-30. *Willow Island Main Lock, Side-Port (SP) Lock.*

TABLE 10-10. Lock Coefficients

Value	Fill	Empty
d_f, d_e, ft	1.00	0.90
C_L	0.80	0.72

TABLE 10-11. Distribution of Total Head Loss

Location	Total Head Loss, $v^2/2g$
Filling	
Upper pool to valve	0.45
Through open valve	0.10
Valve to lock chamber	1.05
Emptying	
Lock chamber to valve	0.93
Through open valve	0.10
Valve to lower pool	0.90

charge coefficients. However, when valve times of 4 min or slower are required for satisfactory operation of the sidewall port system, port system advantages of the more favorable discharge coefficients disappear as the bottom longitudinal system is relatively insensitive to valve speed and a fast valve time can be used at all lifts. Furthermore, permissible filling-and-emptying times can be decreased by enlargement of the culverts in the bottom longitudinal system; this is not the case in the sidewall port system.

10.15 EXAMPLE DESIGN OF HIGH-LIFT LOCKS

10.15.1 Objectives

The primary objectives in the design of a lock filling-and-emptying system are rapid fill-and-empty cycle; safety to vessels, structures, and personnel; economic construction; minimum maintenance; and smooth uninterrupted operation.

10.15.2 Turbulence

The system must be designed so that turbulence and/or surging in the lock chamber does not cause excessive forces on hawser lines used to secure large vessels or create hazards to smaller craft that could be unmoored.

Excessive surging could result in forces large enough to break mooring lines, causing damage to the service gates and vessel and endangering operating personnel. Comparison of model tests and prototype observations has shown that when a lock is designed so that certain hawser forces are not exceeded in a model, the prototype will be satisfactory for the moored vessels as well as small craft. These limiting hawser forces as measured in a model are 5 prototype tons (short tons) for barge tows and 10 prototype tons for single vessels (ships) up to 50,000 prototype deadweight tons. Hawser forces for larger vessels are allowed to exceed 10 tons, since they will be required to have more mooring lines than smaller vessels.

10.15.3 Flow

For high lifts, the flow into the lock chamber must be equally distributed if objectionable turbulence and hawser stresses are to be avoided while accomplishing acceptable filling times. Through a series of model tests of specific projects (Table 10-12) and general studies, a balanced flow system has been developed for various locks. This system eliminates the surge and oscillation inherent in the sidewall port culvert and end filling systems by distributing flow uniformly throughout the lock chamber. During filling of the lock when the filling valves are open and the emptying valves are closed, flow enters culverts in each sidewall through intakes in the upper pool and is carried to the midpoint of the lock chamber where it is equally divided and directed to the upstream and downstream ends of the chamber. Flow in each end of the lock chamber is then divided into distribution culverts and discharged through a manifold of small ports into the lock chamber. During emptying of the lock when the emptying valves are open and the filling valves are closed, water from the lock chamber enters the distribution culverts through these small ports and is carried to the midpoints of the

TABLE 10-12. Lock with Balanced Flow Filling-and-Emptying Systems

Name	Location	Lift	Lock Chamber Size
Bankhead	Warrior River, AL	69 ft	110 ft × 670 ft
Bay Springs	Tennessee–Tombigbee Waterway, MS	86 ft	110 ft × 670 ft
Bonneville	Columbia River, OR	69 ft	86 ft × 675 ft
Lower Granite	Snake River, WA	105 ft	86 ft × 675 ft
Trinity River (proposed)	Trinity River, TX	60 ft	84 ft × 655 ft
Walter Bouidin (proposed)	Coosa River, AL	130 ft	84 ft × 630 ft

lock chamber where it is equally divided into the sidewall culverts and discharged into the lower pool.

10.15.4 Crossover Culverts

The portion of the system near the midpoint of the lock where flow from each wall culvert is divided and directed to the ends of the chamber is designated the crossover culverts. The following methods of dividing flow have been used:

a. The side-by-side culvert method where flow is divided by a vertical wall (Figure 10-31).
b. The over-and-under culvert method where flow is divided by a horizontal splitter (Figure 10-32).

The over-and-under crossover culvert (horizontal flow divider) is preferred because it provides a more stable distribution of flow and is less likely to result in cavitation. Also, this method is more hydraulically efficient than the side-by-side method. In fact, the only reason for using the side-by-side method would be the cost advantage that may result under certain foundation conditions because the over-and-under crossover requires more depth to construct.

10.15.5 Divider Piers

The divider pier is an important feature of the side-by-side crossover culvert because it provides a means for directing 50% of the flow to each end of the lock chamber and results in more stable flow conditions through the crossover culverts. However, this area is subject to cavitation that can occur in cores of vortices shed from the divider piers with high lifts. Therefore, this method of division is not recommended with lifts greater than 60 ft.

10.15.6 Combining Culverts

With either crossover culvert system, flows from the two wall culverts discharge into a common culvert in each half of the lock so that the entire distribution system will be used even though only one wall culvert is in operation. These are called combining culverts. A relatively constant cross-sectional area is maintained from the wall culvert through the crossover and combining culverts. With the over-and-under crossover culvert system, combining of flow is accomplished as shown in Figure 10-32, and with the side-by-side crossover culvert, combining of flow is accomplished as shown in Figure 10-31. With the latter system, distribution of flow in the combining culvert with only one wall culvert operating is very sensitive to the location

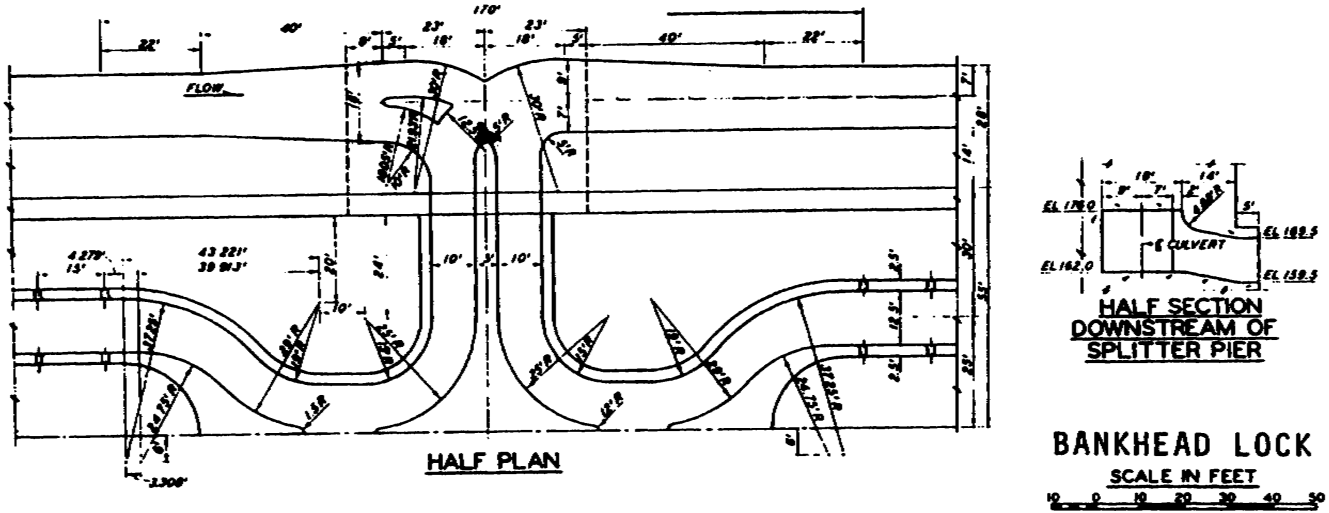


FIGURE 10-31. Balanced Flow Filling and Emptying, Side by Side Crossover Culverts with Two Distribution Culverts in End of Lock.

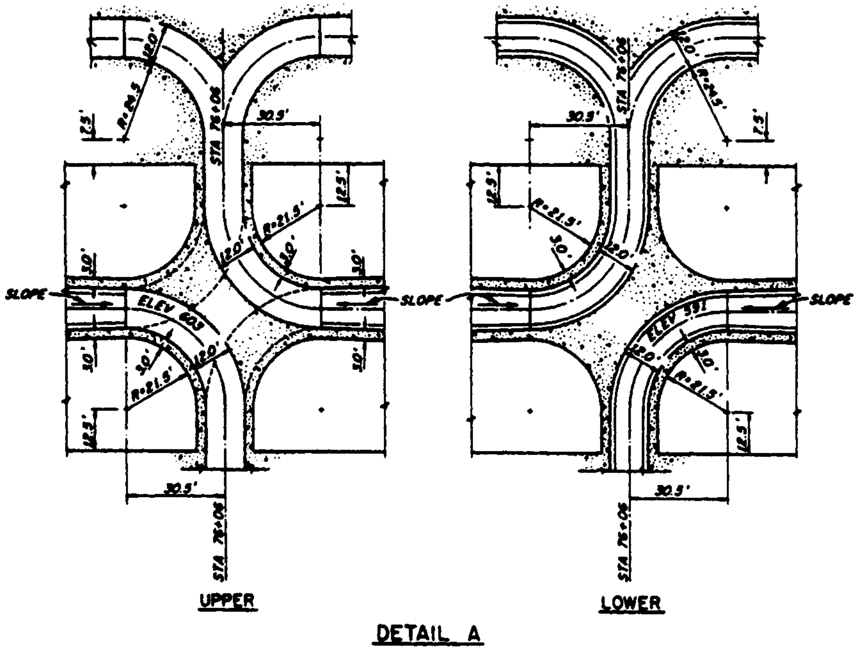


FIGURE 10-32. Balanced Flow Filling and Emptying, Over and Under Culverts with Two Distribution Culverts in Each End of Lock.

of the downstream edge of the separation pier. If the downstream edge of the pier is too short, excessive flow passes to the side of the combining culvert opposite the five culvert; if too long, excessive flow remains on the side of the combining culvert adjacent to the active culvert.

10.15.7 Distribution Culverts

From the combining culvert, flow is redivided into 2 or 4 distribution culverts in each end of the lock as shown in Figures 10-31 and 10-32. The exact conditions under which two or four distribution culverts are needed have not been clearly established, but this depends upon lift, culvert size, and lock chamber length-to-width ratio. In the Bankhead Lock and Bay Springs Lock, 2 distribution culverts in each half of the chamber were adequate. In a series of general tests with a 110 by 1,270 ft lock, 4 distribution culverts were required. Thus, with a length-to-width ratio of 6.1, 2 distribution culverts were adequate, but with a length-to-width ratio of 11.5, 4 distribution culverts were required. In the Lower Granite Lock, with a length-to-width ratio of 7.9, 4 distribution culverts were used. For locks proposed on the Trinity River, length-to-width ratio of 7.8, model tests showed that 2 distribution culverts were adequate, but the maximum lift was only 60 ft. In the proposed Walter Bouldin Lack with a lift of 130 ft and a length-to-width ratio of 7.5, 2 distribution culverts produced satisfactory hydraulic conditions in model tests.

10.15.8 Cross-Sectional Area

Certainly, the four distribution culverts result in a more symmetrical flow pattern in the chamber than do two culverts, but it also is a more costly system with increased hydraulic losses. Regardless of whether two or four distribution culverts are used in each end of the chamber, it is desirable for the combined cross-sectional area of these culverts to be greater than the cross-sectional area of the wall culverts. This not only has a favorable influence on filling-and-emptying times, but also reduces bursting pressures during filling and collapsing pressures during emptying in the crossover and combining culverts.

10.15.9 Port Manifolds

In each of the distribution culverts a manifold of ports discharges flow into the lock chamber. These ports extend over at least 50% of the length of the chamber. In designs with 4 distribution culverts (one pair in each end) the port manifolds are centered on the ¼ and ¾ points of the chamber, and each manifold extends over at least 12.5% of the total length of the lock. The size of the ports ranges from 4.20 to 6.28 ft^2. A port area to distribution culvert area ratio of approximately 1.0 results in good distribution of flow in the lock chamber. Port spacings of 14 to 18 ft were used in the various designs discussed earlier and spacing appeared to have very little effect on flow conditions. The prime objective in port spacing is to use as much available length of the lock chamber as possible.

10.15.10 Baffles

A large portion of the energy of the jets issuing from the ports is dissipated in turbulence in trenches along the distribution culverts. Baffles on the walls of the trenches are used to prevent upwelling of the jets from the ports.

10.15.11 Bottom Filling and Emptying

The bottom longitudinal filling-and-emptying system unquestionably is the best system developed to date for high-lift locks in the United States. The locks that have been built using this system have operated very efficiently with very little turbulence in the lock chamber. For example, the Lower

Granite Lock fills in about 8.1 min with a lift of 105 ft and the Bankhead Lock fills in 7.7 min with a 69 ft lift. The water surface in both of these locks is extremely smooth during the entire filling cycle. Model tests indicate that the Bay Springs Lock will fill in about 8.3 min with a lift of 86 ft.

10.15.12 Reverse Tainter Gates Valves

The filling-and-emptying culvert valves of high-lift locks are very important in the overall design of the system. Reverse tainter gates have been used as the control valves in high-lift locks recently constructed in the United States. When a large volume of air is drawn into the culverts, the air may pass through the ports and erupt in the lock chamber. The resulting disturbances would be hazardous. By reversing the tainter gates, that is, placing the trunnions upstream from the skin plate and sealing against the downstream end of the valve well, air is prevented from entering the culvert at the valve recess during the opening period if the pressure gradient drops below the top of the culvert.

10.15.13 Tainter Valves

Three structurally different types of reverse tainter valves (horizontally framed, double skin plate, and vertically framed) have been used in the United States. The horizontally framed valve is desirable structurally, but the double skin plate and vertically framed are less susceptible to critical hydraulic loads and load variations during the opening cycle.

10.15.14 Cavitation

Prevention of cavitation downstream from the valves is a very difficult problem for designers, particularly as lifts increase to values greater than 100 ft. High velocities and low pressures are induced as flow accelerates immediately downstream from the valves during the valve opening period. In those instances, the local flow acceleration is sufficient to lower the local pressure to the vapor pressure of water and form cavities within the flow. These cavities collapse rapidly or implode either in the water or against the downstream boundaries as they enter the increased pressure that results from the decreased velocity of flow as it expands and decelerates in the culvert downstream of the valve. This has resulted in lockmasters reporting loud pounding noises indicating cavitation implosions within the flow. In some instances, these booms have been violent enough to shake the lock walls and break windows. The implosion of the cavities against solid boundaries results in rapid pitting or damage to valves and appurtenances and to the concrete culverts.

10.15.15 Pressures

In some designs, pressures low enough to cause cavitation are avoided by submerging the culvert at the location of the valve so that the pressure gradient is maintained above the top of the culvert. However, as lifts increase, it becomes increasingly costly to provide adequate submergence. Through prototype tests at some of the high-lift locks on the Columbia River it was found that admitting a controlled amount of air into the culverts at each valve virtually eliminated the pounding noises. Air was drawn through a vent placed downstream from the valve into the culvert system during the valve opening period, was entrained as small bubbles in the highly turbulent flow, and emerged in the lock chamber so entrained that it merely caused the water to look milky. It was concluded that the air cushioned the collapse of the large cavities, eliminated shock pressures, and thus eliminated the pounding noises. This procedure allowed the culverts to be placed at a much higher elevation, thus minimizing excavation costs. Several locks have been constructed in the United States using this procedure, and no operation difficulties or hazardous conditions have been reported where pressures on the culvert roof were low enough to draw air during the valve opening period.

10.15.16 Culvert Expansions

Through model tests it was found that expanding the culvert roof upward downstream from the valve (Figure 10-33) would increase pressures on the roof of the culvert just downstream from the valve. Also, in these tests it was found that the location of the expansion with respect to the valve directly affected the pressure on the roof of the culvert in the area immediately downstream from the valve. Thus the use of expansions downstream from the culvert valves is a very practical means of controlling the pressures and allowing the valves to be set at a more economical elevation.

Expansions started at locations immediately downstream from the valve to a distance of 6.5 times the valve height (Figure 10-33). Valve energy loss coefficients are essentially the same with no roof expansion, and with roof expansions beginning 4 and 6.5 times the valve height downstream from the valve. Thus culvert expansions that begin 4 valve heights or more downstream have no effect on the loss coefficient for valve openings of 30% or greater. Expansions beginning within a distance of 4 valve heights of the valve increased energy loss coefficients as the expansion was placed closer to the valve.

10.16 MECHANICAL LIFTS

10.16.1 General

In Germany, France, and Belgium, structures have been built to transfer vessels from one water level to another without using navigation locks.

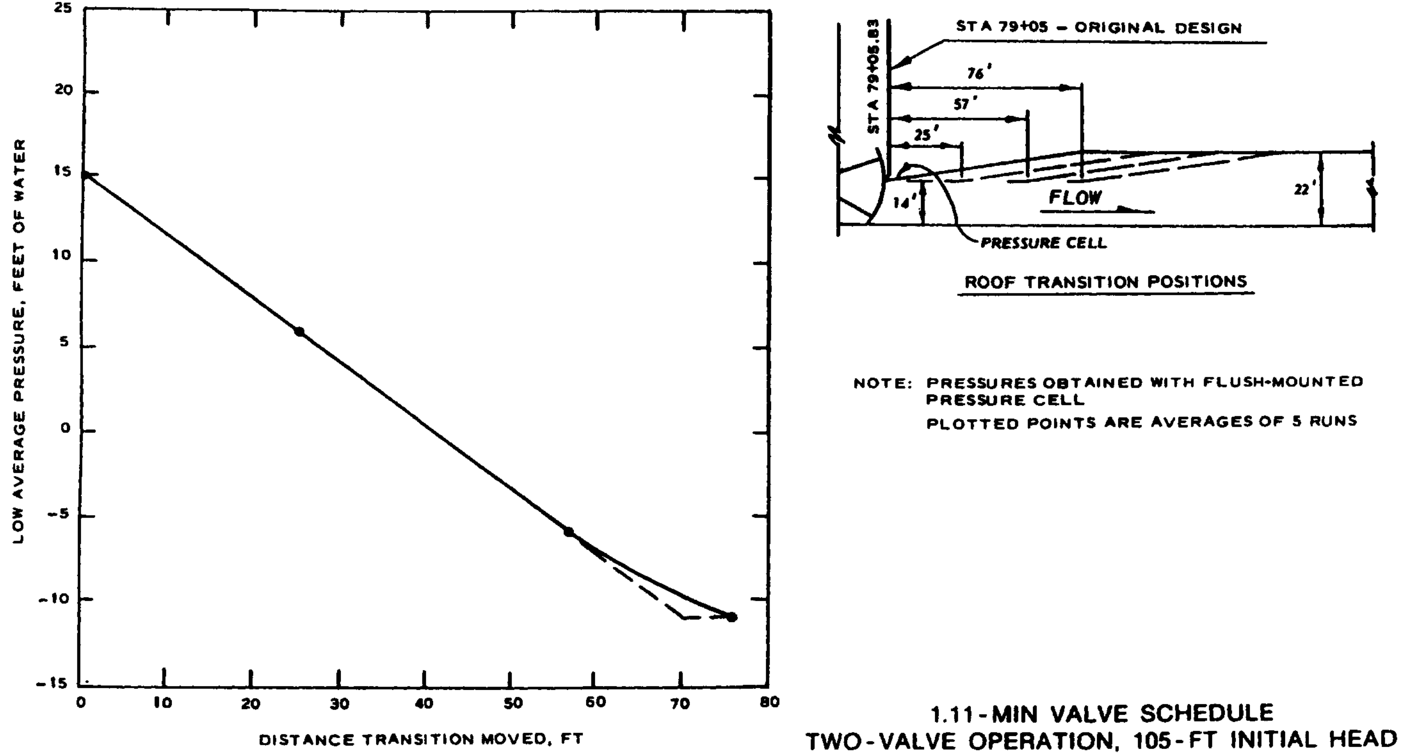


FIGURE 10-33. Effects of Culvert Expansion on Valve Pressure Drop.

These structures are known as mechanical lifts and move a vessel either vertically upward and downward, or upward or downward on an inclined plane. In the early part of the 19th century there were two or three such devices in the US. However, these "lifts" were quite small and involved moving a small canal boat up an inclined track on a wheeled truck arrangement.

10.16.2 Types

Ship elevators or vertical mechanical lifts have been built in Germany to serve in place of locks where lockage water was not available. One of the first ones was built on the Dortmund–Ems Canal at Henrichenburg in 1899. This mechanical lift was used until 1962 and was then replaced with a new one. It consists of a rectangular tank with a gate in each end. This rectangular tank is supported on two air-filled flotation chambers that move up and down in water-filled shafts underneath the structure. The flotation chambers provide enough buoyancy to balance the weight of the water-filled tank. When the rectangular tank (trough) is at the lower canal level, it connects directly to the end of the canal. The gate in the trough is lowered, the gate in the end of the canal is lowered, and a vessel waiting in the canal enters the trough. When the vessel is moored in the trough, the gates in the ends of the canal and the trough are closed. The trough is raised to the upper canal level by means of motor-driven threaded vertical shafts running through nuts attached to each corner of the trough. Rotation of the shafts is synchronized, and the trough remains level at all times. Since the flotation chambers provide an upward force equal to the weight of the tank, the threaded shafts have only to overcome mechanical friction and control movement of the tank. When the trough is secured to the end of the upper canal, the gates in the ends of the trough and the canal are opened, and the vessel can depart. Transfer of a vessel from the upper canal to the lower canal is accomplished in a similar fashion. The trough is 295 ft long, 39 ft wide, and 10 ft deep, and can transit a 1,500 ton (2,200 pound ton) vessel in about 30 min. The difference in elevation is about 46 ft.

Inclined plane mechanical lifts of two different designs have been built in Belgium and France. In the French project a water-filled tank or trough moves sideways up and down on rails on an inclined plane. The action is similar to the Henrichenburg lift, except the trough moves up an incline instead of vertically, and the dead weight of the trough (plus water) is offset by counterweights moving in trenches on the incline. The French project is located on the Rhine–Marne Canal near Arzviller, France. The trough moves through a vertical distance of 44 m (144 ft) over a horizontal length of about 100 m (328 ft). It is designed for 300 ton vessels (2,200 pound tons) and replaces 17 very old, small canal locks.

The Belgian inclined plane lift is located in the Brussels–Charleroi Canal. There are two separate parallel tracks at this lift, and the troughs move up endways. The horizontal length of the incline tracks is 4,700 ft, and the vertical distance of the incline is about 220 ft. The travel time for the trough is 20 min. Allowing for a total entry and startup time of 5 to 10 min and a stopping and exit time of 5 to 10 min, the total transit time would be about 35 to 40 min. The two troughs operate independently and have dimensions of 285 by 39 by 10 ft. Each trough can carry one 1,350 ton (2,200 pound ton) vessel, which has almost the same carrying capacity as a 1,500 ton (2,000 pound ton) barge in the United States.

10.16.3 Capacity

If three conventional 110 by 600 ft locks were used to overcome the 220 ft difference in elevation, each lock would have a lift of about 73 ft. The transit time for an 8-barge tow through each of these locks would be about 25 min. Adding 15 min for travel time between locks (assuming the locks are 2,500 *h* apart) gives a total travel time of 90 min (1.5 hr) to transit 12,000 short tons. To transit 12,000 tons through the incline (moving in the same direction) would require 4 trips for each 2 troughs, which would total 5.33 hr (8 × 40 min). Thus the net total transit time required to move 12,000 tons through the incline is 3.5 times greater than the time required to move 12,000 tons through 3 locks and travel a distance of about 5,000 ft. Moreover, the lock system would have more than 3.5 times the capacity of the incline, because all 3 locks would not be in use by 1 tow at the same time.

10.16.4 Water Slopes

French entrepreneurs have developed and patented a system wherein a wedge-shaped volume of water is pushed up or down a sloping rectangular channel with a vessel floating in a wedge of water. A "water slope" (Figure 10-34) is located at Montec, France. The rectangular channel is 20 ft wide, is on a 3% slope, and will accommodate a 300 ton vessel with a 7 ft draft. The entire structure replaces 5 old locks.

The system apparently performs very well in the present situation, but to be commercially feasible in the US the channel would have to be 4 to 5 times wider, the walls would have to be several times higher in order to provide adequate depth for a 580 ft tow, and structural design problems would be extremely complex for the greater sizes. The system could not possibly be energy efficient.

10.16.5 SEPARATE FACILITIES FOR RECREATIONAL CRAFT

At places where recreational craft appear in considerable quantities, the introduction of separate handling facilities may be worthwhile. This is par-

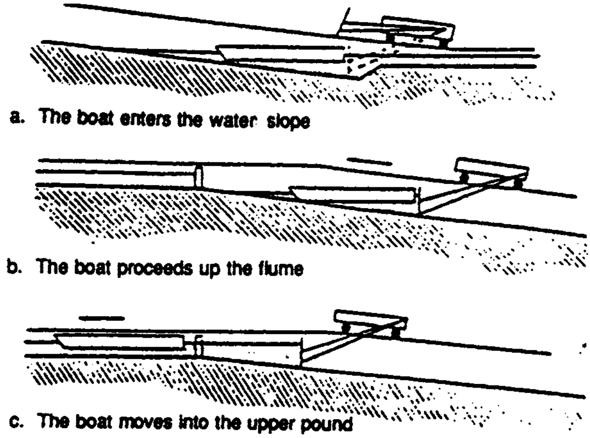


FIGURE 10-34. Water Slope.

ticularly true when the period of peak recreational demand corresponds to the period of peak commodity movement. Such separate facilities could be canvas slings or steel tanks to lift the craft from one level to another, separate small locks out of the main navigation channel, or an inclined plane moving lock such as has been used in Europe and in early US canal development. Separation of recreational traffic from towboat traffic would also appear to be a safety improvement.

Analyses of alternative small craft lifts were considered at Kentucky Lock and indicated that the inclined plane type would be more feasible from the standpoint of economics and operation. The inclined plane would be laid out on a steel superstructure that would carry the tracks on a uniform grade up the downstream side of the embankment to an elevation permitting adequate clearance over the railroad and highway. The superstructure would then convey the tracks across the top of the dam to a similar inclined plane on the upstream side. The boat would ride in a tub that would accommodate one craft 24 ft or less in length.

Twenty of the Upper Mississippi River locks have partial provisions for a second lock chamber, 100 ft by 360 ft. These provisions include an upper gate sill, upper portion of the river wall, and recesses in the intermediate wall for the lower miter gate and gate machinery. Completion of this lock chamber would involve damming and dewatering the chamber area; removing accumulated debris and providing scour protection measures; constructing the river wall and chamber floor; removing and rehabilitating the upper miter gate; and installing gates, valves, operating machinery, and appurtenances. Commercial traffic would also be able to use the new lock if the main chamber failed.

Eighteen of the 20 Upper Mississippi River locks with partial provisions for a second lock chamber include either a roller or tainter flood control gate adjacent to the river wall. At these 18 locks, the completion of a 400 ft auxiliary lock would be possible. The 400 ft chamber would be built by extending the river wall, Dam Pier 2, and possibly the intermediate wall downstream. A new miter gate and tainter gate would be built in a monolith at the lower end of the chamber. The wall and pier extensions would be made from steel sheetpile cells. The extension of the dam pier and any extension of the intermediate wall would be a solid cell wall. The river wall would be steel sheetpile cells spaced with 10 ft clearances between cells. The monolith would be keyed into the intermediate wall and the dam pier extension. The area between the river wall and the dam pier extension would function as a flume to fill the lock chamber (the area between the river wall and the intermediate wall). Commercial traffic would be able to use the new chamber if the main chamber failed.

A mobile floating lock is a self-contained, fully operational lock structure that can be positioned behind the existing upper miter gates for the auxiliary chamber. This device would be approximately the size of 3 barges abreast (105 ft by 200 ft). The lock is a steel vessel similar to a dry dock. The sides would be floating tanks housing the operating machinery and controls. The upper and lower gates, integral parts of the dock, would be permanently mounted within the outside tanks. The upper and lower gate types have not been determined but would probably be submerging tainter gates or hinged drop gates, depending on the available depth in the chamber. Filling and emptying would be done through ports in the chamber floor.

The small-scale steel lock, 25 ft by 80 ft, would be a double-wall steel structure of $^{3}/_{8}$ in. plate with adequate diaphragms. The upper gate bay would include a vertical lift gate and an emptying system. The upper sill elevations would be set to accommodate sailboats up to 40 ft long.

The 25 ft by 80 ft concrete and sheetpile lock would be a concrete U-frame structure on a sand foundation. The structure would include a concrete upper gate bay monolith, a lower concrete gate bay monolith, and a lock chamber of sheetpile walls with a revetment floor. The inside face of the cofferdam would act as the outer form for the concrete gate bay monoliths and would be constructed on site.

The differential railway lift consists of a steel tank (pan) carried up an inclined plane, over a crest, and down a reverse plane without being tilted. The pan is rigidly suspended from a carriage equipped with two sets of wheels to travel on a system of track elevated over the earth dike. The outer set of wheels maintains the plane horizontally while the carriage travels above the downstream face of the dike on a 2.5H to 1V incline. The inner set of wheels maintains the pan horizontally while the carriage travels above the upstream face of the dike on a reverse 2.5H to 1V incline. Both sets of wheels are used as the carriage travels above the crest on a double set of differential rails.

The steel tank on inclined rails consists of a steel tank (pan) supported by an overhead crane at each corner. The cranes lift the tank vertically out of the water, travel horizontally along rails across the dike, and then lower the tank into the water on the other side. The crane trolleys on each rail are structurally separated from the trolleys on the other rail and each uses one drive wheel. The four lift motors and both crane drives are electrically synchronized, eliminating overhead clearance restrictions.

The mobile boat carrier system is based on a mobile boat carrier presently used for launching certain pleasure craft. The slings could be replaced with a tank (pan) for holding the boats being transported. The modified boat carrier would lift the tank out of the water, travel along a horizontal track across the dike, and lower the tank into the water on the reverse side. The carrier cross-member would restrict the overhead clearance. Additional studies would be required to determine if the slings could be safely adapted to various boat shapes.

The inclined channel lift is similar to a device in operation at Montech, near Toulouse, France, connecting two canals. Two water levels in the canal are joined by a 480 ft flume or concrete ramp having a U-shaped section. Water at the upper level is held back by a tilting gate. The boat on the lower level enters the approach basin. A large plate at the end of two arms is lowered into the water behind the boat, forming a wedge-shaped body of water on which the boat floats. The plate is then pushed forward by two 1,000 horsepower diesel-electric locomotives, one on each bank.

The inclined plane lift resembles Belgium's Ronquiaes ship lift located near Brussels. This single structure is 4,700 ft long and raises and lowers craft 225 ft. Two inclined planes raise and lower 1,500 ton barges 225 ft in 22 min. Barges enter a tank (pan) with gates at either end and are pulled or towed by 6 125 kilowatt electric motors connected to the tanks by 2.25 in. diam cables. When loaded, the tanks weigh between 5,500 and 6,280 tons. Counterweights weighing 5,733 tons run up and down in recesses between the tank rails. The tanks measure 49 ft by 300 ft and are 14 ft deep. Both tanks and counterweights ride on spring-suspended wheels that run on steel rails.

The version considered for the Upper Mississippi River would have one tank approximately 26 ft by 80 ft and maintain a depth of about 4 or 5 ft. The system would be operated by remote control from the main lock and monitored by television and 2-way radio communication.

10.17 SOURCE

The majority of information was taken from US Army Corps of Engineers Manual EM 110-2-1604, *Hydraulic Design of Navigation Locks*. Another excellent source of information on lock design is a September 1984 report by John P. Davis entitled *Hydraulic Design of Navigation Locks*.

Chapter 11
DAM DESIGN

11.1 GENERAL

Navigation dams can be relatively high structures, such as those on the Columbia and Snake Rivers. However, most navigation dams are low-head structures. Their basic purpose is to provide adequate depths for navigation during low-flow periods and to offer minimum resistance to high flows. This chapter concentrates on the design of spillways for low-head dams. The following guidance is mainly a result of analysis of specific low-head navigation projects. A definition sketch is given in Figure 11-1.

11.2 CREST DESIGN

11.2.1 General

Since the project is planned to offer minimum resistance to flood flows, the fixed portion of the spillway must occupy only a small part of the cross-section of the river channel. Thus a gate sill with its elevation at or near the elevation of the streambed is required and damming during low flows must be accomplished by movable gates. The lower the head on the crest, the lower is the unit discharge. This results in a longer crest but lesser requirements for the stilling basin and downstream channel protection. Conversely, the higher the head on the crest, the higher is the unit discharge. This results in a shorter crest length but greater requirements for the stilling basin and downstream channel protection. Many low-head navigation dams operate under highly submerged flow conditions. The discharge coefficients for a low, submerged, broad-crested weir are close to those for a similar low, submerged ogee crest. With a low gate sill an ogee crest may not provide sufficient space for operating gates and bulkheads. Thus, for these

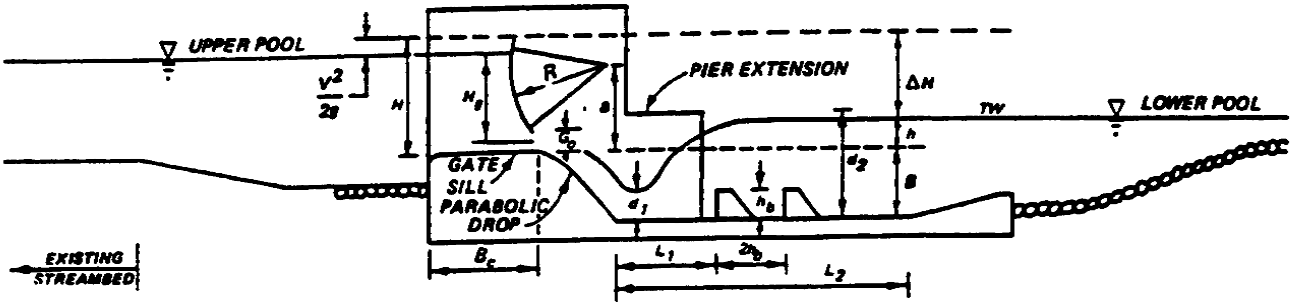


FIGURE 11-1. *Definition Sketch of Typical Navigation Dam.*

reasons, a broad-crested weir is often indicated and structural requirements usually dictate the width of the crest to be approximately the same as the damming height of the gates.

11.2.2 Upstream Face

Although a vertical upstream face slope has been used on most low-head navigation dams having a broad-crested weir, other slopes can be used. The minimum radius connecting the upstream face with the horizontal portion of the broad-crested weir should be as follows.

Head (ft)	Radius (ft)
<20	3
20–30	4
30–40	5
40–50	6

11.2.3 Downstream Face for Nonsubmersible Gate Spillway

The downstream face of the weir can be shaped so that flow under partially opened gates will adhere to this face of the weir and thus move to the floor of the stilling basin where it can be dispersed by baffles and/or the end sill. If the downstream face breaks away from the weir crest too sharply, the nappe will separate from the weir, and an eddy in a vertical plane will form under the nappe in the upstream portion of the stilling basin. Under certain tailwater conditions, this eddy will force the nappe upward and then it will dive through the tailwater and attack the exit channel downstream of the stilling basin. This undesirable type of action, known as an undulating jet with a free nappe, generates severe surface waves. Of course, economics dictates that the horizontal extent of the downstream face of the weir be minimum. A free jet leaving the horizontal weir crest will follow the path,

$$X^2 = \frac{2V_0^2 Y}{g} \tag{11-1}$$

where

X, Y = horizontal and vertical coordinates
V_0 = initial free jet in feet per second (ft/sec) = $\sqrt{2gH}$
g = acceleration due to gravity in ft/sec^2
H = upper pool elevation, crest elevation

However, the nappe will adhere to the downstream face if V_0 is the theoretical velocity resulting from only one third of the actual head. Thus, if the upper pool is 36 ft above the weir crest (H = 36 ft), V_0 for determination of the shape of the downstream face of the weir should be based on a head of only 36/3 or 12 ft. That is, $V_0 = \sqrt{2g(12)}$ = 27.8 ft/sec; and the equation for the downstream face should be about $X^2 = 48Y$. Since the range of data used to develop this relation is limited, the steepest trajectory that should be used is $X^2 = 40Y$. For heads greater than 40 ft, model testing is required. Using one third of the head on the crest results in a downstream face shape which is close to that resulting from the procedure used for high spillways (presented in EM 1110-2-1603). The techniques presented in EM 1110-2-1603 can be used for heads greater than 40 ft. The trajectory resulting from using one third of the head on the crest is the steepest that can be used without severe negative pressures occurring on the downstream face; flatter trajectories can be used. The parabolic trajectory continues to the stilling basin floor unless terminated by a constant slope which may be desired for ease of construction. A slope of 1V:1H was used below the parabolic trajectory in the investigation of pressures on the downstream face of the crest. Examples of different crests are shown in Figure 11-2. Downstream faces having "steps" have been used on Mississippi River Locks and Dams Nos. 5A, 6, 7, 8, and 9. These structures have relatively small differentials (5.5 to 11.0 ft) between upper and lower pool elevation.

11.2.4 Downstream Face, Submersible

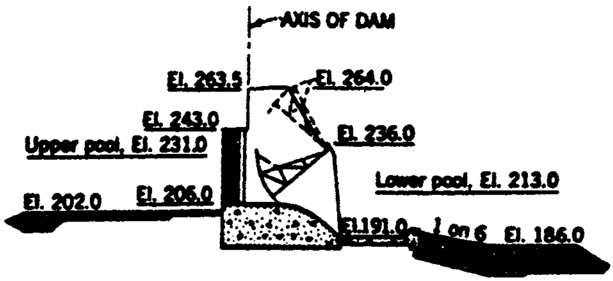
Submergible tainter gates are used to pass ice over the top of the gates. As shown in Figure 11-3, submersible tainter gates can be either the "piggyback" type or those in which the crest shape allows the bottom of the tainter gate to drop below the flat portion of the crest. The piggyback type uses the parabolic trajectory given in Section 11.2.3. Two examples of the downstream crest shape for the second type of submergible tainter gate are shown in Figure 11-3. Gate bays for submergible gates should not be so wide that undesirable gate vibrations develop.

11.2.5 Intersection of Downstream Spillway Face and Stilling Basin Floor

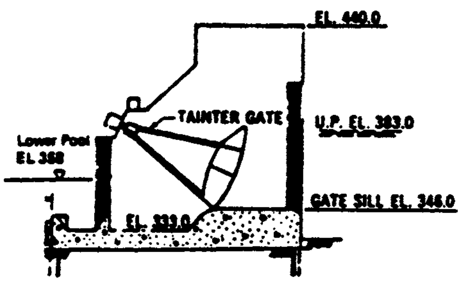
Toe curves at the intersection of the downstream spillway face and the stilling basin floor are not widely used in low-head navigation dams. Guidance for toe curve pressures below ogee crests is given in HDC 122-5.

11.2.6 Crest Pressures, Velocities, and Water Surface Profiles

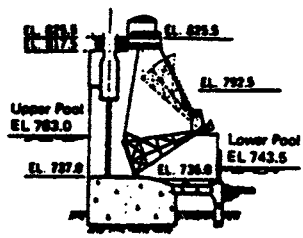
For most low-head navigation dams, velocities occur at small gate openings when the effective head is high and tailwater level is low. The latest design policies require that under emergency conditions, any one gate can be fully opened without causing severe erosion damage to the downstream



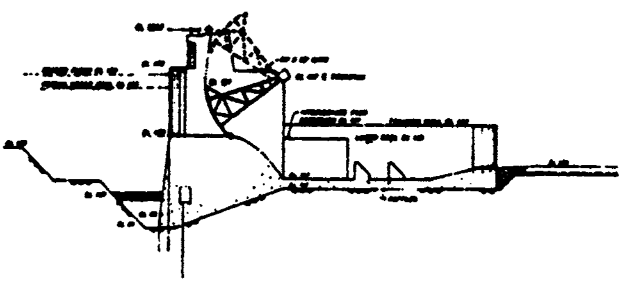
DAVID D. TERRY LOCK & DAM (NO. 6)
(ARKANSAS RIVER)



CANNELTON LOCKS & DAM
(OHIO RIVER)

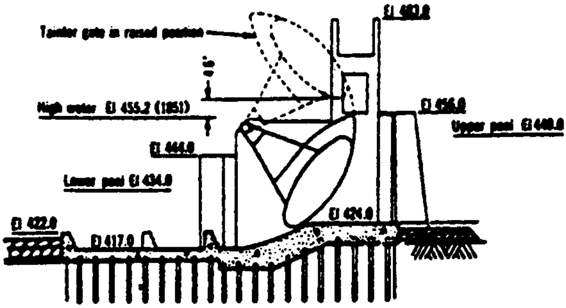


MAXWELL LOCK & DAM
(MONONGAHELA RIVER)

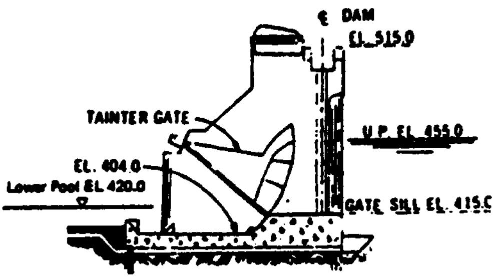


COLUMBUS LOCK & DAM
(TOMBIGBEE RIVER)

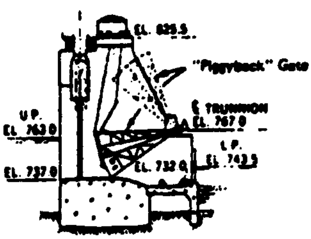
FIGURE 11-2. Examples of Crests, Nonsubmergible Gates.



LOCK & DAM 24
(MISSISSIPPI RIVER)



MARKLAND LOCKS & DAM
(OHIO RIVER)



MAXWELL LOCK & DAM
(MONONGAHELA RIVER)

FIGURE 11-3. *Examples of Crests, Submergible Gates.*

scour protection measures. Flow velocities and pressures should be determined for both of these operational conditions. The velocities are needed to assign appropriate tolerances for construction of the spillway surfaces. Pressures resulting from these velocities are needed to ensure against cavitation conditions and also to determine the uplift forces needed by structural designers to check the spillway stability. Crest pressures and water surface profiles have not been measured for a wide range of heads, gate openings, approach elevation, apron elevations, and the like. Available information is provided in EM 110-2-1605.

11.3 SPILLWAY CAPACITY FOR HIGH-HEAD DAMS

Spillways for high-head navigation dams are generally designed with adequate capacity to pass the PMF flows. At this condition, all flows would still be limited to the spillway section; adjacent concrete or embankment structures would have adequate freeboard to prevent overtopping of these structures. In some cases, stilling basin designs would be based on the PMF condition, but in other cases tailwater buildup for this discharge would drown out the hydraulic jump and the design would be based on some lesser discharge condition. See reference EM 1110-2-1603 for determining spillway capacity for high-head dams.

11.4 SPILLWAY CAPACITY FOR LOW-HEAD DAMS

Typically, low-head navigation dams are designed to pass flood flows utilizing not only the main spillway section normally located within the river channel but also supplemental spillways located across the overbanks and even the lock access road and esplanade areas. The width and potential carrying capacity of the overbanks will affect the main spillway capacity. However, the objective in sizing the main spillway is to minimize the headwater–tailwater differential at the time flood stages exceed the riverbanks, extend out into the overbank areas, and begin overtopping the uncontrolled spillways. The smaller this head differential, the less will be flood stage increases over preproject conditions, and the simpler will be the scour protection measures required for the overbank uncontrolled spillway sections. These head differentials can be kept low by providing a main spillway capacity roughly equivalent to the natural river capacity at the project design flood. Providing this much capacity can be difficult on smaller rivers because the navigation lock must be prominently located within the main river channel to provide safe lock approach conditions. Consequently, low-flow dam spillways frequently extend well into the bank line opposite the lock, unless the lock is located within a navigation canal separated from the natural river.

Locating the spillway in a bypass canal is another means of reducing the head differential.

11.4.1 Spillway Crest Elevation

Low-head gated spillways typically have crest elevations set near the riverbed elevation to maximize capacity. Of course, riverbed elevations generally vary across the proposed spillway section. Furthermore, bed elevations in alluvial rivers vary with discharges. An understanding of these alluvial characteristics during flood conditions is required to select the optimum crest elevation. If selected too high, the spillway will be wider than necessary. If selected too low, the discharge control will shift from the spillway crest to an approach channel section when the gates are fully opened; the spillway gates will be higher than necessary; the spillway structural stability will be more difficult to attain; and during low-flow periods sediment will deposit on the spillway thereby hampering gate operations and increasing wear and tear of the gates. At Lock and Dam 4 on the Arkansas River, the spillway crest was set at two elevations with the deeper section next to the lock and the higher section at the opposite bank line where under pre-project conditions sediments normally deposited. After over 15 years of operation, the benefits of the stepped crest are considered negligible, and a constant crest elevation would be recommended. The stilling basin design for multilevel crest elevations is complex.

11.4.2 Overbank Crest Elevation

The spillway crest elevations of uncontrolled overbank sections are generally set as close to the natural groundline as possible to best utilize the natural flow capacity of the overbank areas. However, the overbank spillway should normally be at least three feet above the navigation pool elevation to allow for pool regulation variations, wind setup, and wave runup heights. One exception would be the crest height at a navigation bypass section that is normally just one foot above the navigation pool level.

11.5 POOL-TAILWATER RELATIONSHIPS

The size of the spillway (both horizontal and vertical) affects pool and tailwater elevations. The following general cases can be identified.

11.5.1 Case 1

The dam is of sufficient height that the spillway is not submerged by tailwater for any discharge.

11.5.2 Case 2

The height of the dam is such that the spillway operates continuously or intermittently submerged, but open-river conditions will not obtain at any time.

11.5.3 Case 3

The height of the dam is such that the spillway operates continuously or intermittently submerged with open-river conditions sometimes.

The pool and tailwater elevation regimes (see Figure 11-4) resulting from a particular project (particularly pool elevations) can affect numerous related factors such as the extent of real estate flooded, groundwater table, levee heights, dam and lock wall heights, number and extent of relocations, navigation pass velocities, and so on. Determination of spillway design in relation to these factors is complex, but in general, high narrow spillways are cost-effective, whereas low wide spillways reduce the costs associated with high pool elevations. Sufficient spillway sizes should be studied to optimize overall project costs. Cases 2 and 3 are the most complex due to spillway submergence.

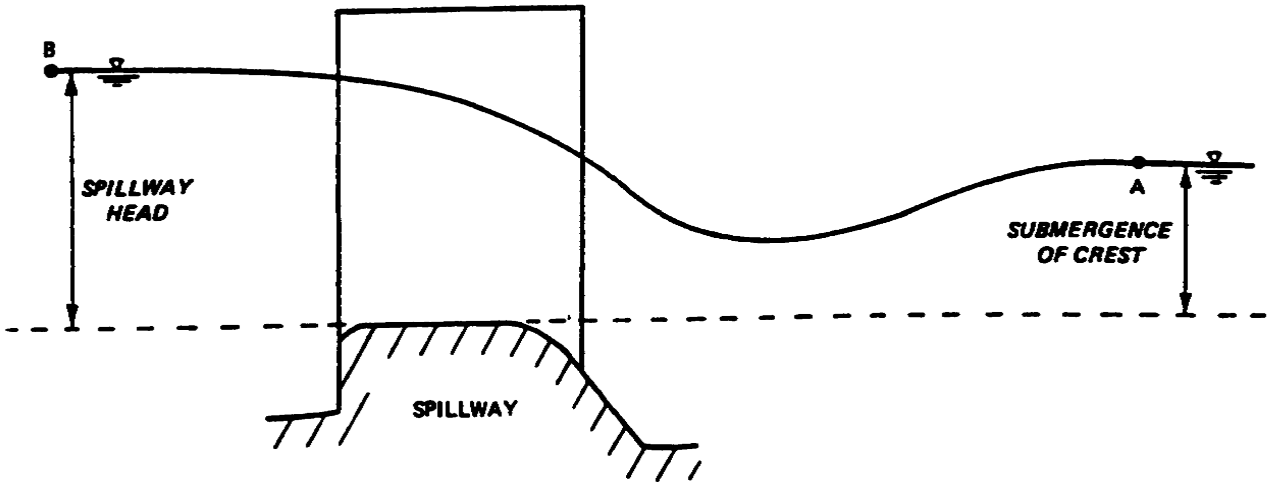
11.6 POOL ELEVATIONS

The complexity of approach flow and interaction with locks, dams, overflow sections, nonoverflow embankments, and spillway submergence make accurate pool elevation determination difficult. This is particularly true when flow approaches spillways at an angle. The d'Aubuisson (see section 11.7) or Kindsvater and Carter formulas can be used for an approximate pool elevation estimate during preliminary submerged spillway design studies.. However, hydraulic models will usually be needed to obtain an estimate of pool and tailwater elevations suitable for detailed design. Computations should be made for the design flood with all gates fully opened and for all operating conditions to establish the maximum upstream pool and backwater profile. Pool elevations and backwater profiles associated with recurrence intervals should also be computed to evaluate real estate, relocations, and other pertinent factors.

11.7 DISCHARGE RATING CURVES FOR GATED, BROAD-CRESTED WEIRS

11.7.1 General

Discharge rating curves are needed for project design and operation. Low-head navigation structures have four possible regimes of flow that



NOTE: POINTS A & B OUTSIDE AREA OF LOCAL DISTURBANCE, DRAWDOWN, ETC.

FIGURE 11-4. *Spillway Head/Submergence.*

result from the effects of the gates and the effects of tailwater on the amount of discharge through the structure. The four regimes are discussed in the following paragraphs and shown in Figure 11-5. Discharge coefficients for low-head navigation dams have been developed mainly for tainter gates. See reference EM 1110-2-1603 for discharge rating of unsubmerged vertical gates or discharge rating of ogee crests. Sufficient data are not available to define the effects of different pier lengths and nose shapes. Model results comparing the ogee and semicircular shapes showed no significant difference for the highly submerged broad-crested weir. Preliminary curves are usually computed from established analytical equations. Physical and mathematical model studies of project facilities frequently include tests to verify both spillway rating curves and flood flow distributions between river channel and overbanks. Model and prototype data from other projects with similar spillway designs are often valuable in refining rating curves. Commonly used equations for preliminary rating curve computations under various spillway conditions are presented.

11.7.2 Determining Flow Regime

EM 1110-2-1603, Figure 5-8, gives guidance to determine the flow regime given headwater H, tailwater h, and gate opening G_0 (definition sketch in Figure 11-1).

11.7.3 Free Uncontrolled Flow

For this flow regime the gates are fully opened and the upper pool is unaffected by the tailwater. The standard weir equation

$$Q = C_F L H^{3/2} \tag{11-2}$$

is applicable and free uncontrolled from discharge coefficients versus Head/ Breadth of Crest are shown in EM 1110-2-1603, Figure 5-9. This curve should be used with caution above $H/B_c = 1.5$. No correction for pier effects is recommended with these coefficients. Crest length should be reduced for abutment effects by the equation

$$L_{\text{effective}} = L_{\text{actual}} - 2KH \tag{11-3}$$

Since the discharge coefficients presented in EM 1110-2-1603, Figure 5-9 already account for pier effects, the contraction coefficient K should be about one half of the value selected from HDC Chart 111.

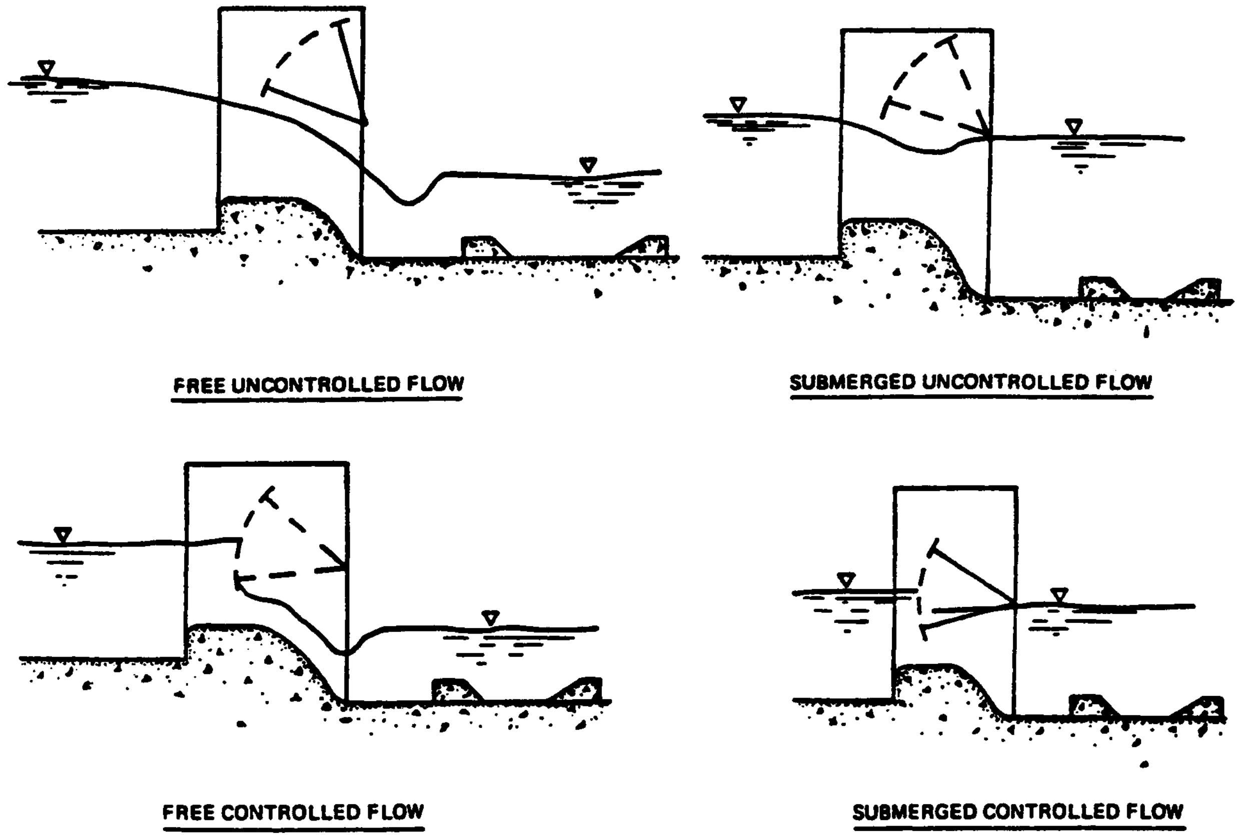


FIGURE 11-5. Four Flow Regimes.

11.7.4 Submerged Uncontrolled Flow

For this flow regime, the gates are fully opened and the discharge is reduced by tailwater conditions. Two procedures are available for determining discharges for uncontrolled spillways under submerged conditions.

(1) Discharge over a submerged weir can be expressed by the equation:

$$Q = C_s L H^{3/2} \tag{11-4}$$

C_s from model data is shown to vary with h/H. Model results show that discharge coefficients for this flow regime are not significantly affected by stilling basin apron elevation. See EM 1110-2-1605 and HDC for discharge coefficients.

Preliminary rating curves for low-head dams under submerged uncontrolled flow conditions can be computed by the d'Aubuisson equation

$$Q = KLh\sqrt{[2g(H-h) + V^2]} \tag{11-5}$$

where

K = spillway coefficient of contraction
L = crest length = number of bays times the bay width, ft
V = spillway approach velocity, ft/sec
H, h = see Figure 11-1

Suggested K values vary with spillway bay width as follows:

Bay Width (ft)	K
40	0.80
50	0.85
60	0.90
110	0.95

These coefficients were developed from experience with prototype structures.

11.7.5 Free Controlled Flow

For this flow regime, the gates are partially open and the upper pool is unaffected by the tailwater. Discharge is controlled by the gate opening and two approaches are available for determining discharge.

(1) For a gross head on the gate less than 30 ft and gate opening less than 14 ft, the following equation is applicable.

$$Q = C_g L G_o \sqrt{2gH} \tag{11-6}$$

Discharge coefficients C_g are found in EM 1110-2-1605, Figure 5-11.

(2) For conditions outside the range covered in (1) above, a comprehensive treatment of the effects of gate location and geometry on discharge for free controlled flow is presented in HDC 320-4 to 320-7. Caution should be used because the equations and symbols are not the same in the two methods.

11.7.6 Submerged Controlled Flow

For this flow regime, the gates are partially open and the upper pool is controlled by both the submergence effect of the tailwater and the gate opening. The applicable equation is

$$Q = C_{gs} Lh \sqrt{2g\Delta H} \tag{11-7}$$

The submerged controlled discharge coefficient C_{gs} as a function of h/G_o for various apron elevations is given in EM 1110-2-1605.

11.7.7 Rating Curve Accuracy

11.7.7.1 Discharge Coefficients. Spillway rating curves as computed by the preceding equations require verification for final designs. Significant errors are possible because of the unique approach conditions at proposed projects. Although data comparing model-prototype rating curves are rare, such information derived from similar existing projects would be valuable for rating curve verification. In finalizing rating curves for major navigation systems, special prototype spillway measurements on similar existing projects should be considered.

11.7.7.2 Tailwater Inaccuracies. Tailwater rating curves are extremely important to the design engineer. The selected tailwater curve will be used in the design of spillway capacity, stilling basins, wall heights, foundation drainage, erosion protection, navigation channel depths, and many other critical elements that make up a total project design. It is imperative that the hydraulic engineer have an accurate estimate of what the tailwater curve will be before, during, and after project construction and throughout the life of the project. The hydraulic engineer must evaluate the likelihood that the tailwater rating will change over this time period and evaluate the extremes to which this change may take place. Furthermore, this information must be passed on to other engineers designing project features so that project integrity will remain intact as the rating curve shifts. The designer is cautioned against spending too much effort in refining inconsequential parameters, such as spillway pier shape coefficients, without paying sufficient attention to potential shifts in tailwater rating curves which can, of course, have drastic influences on submerged spillway capacity. An example of a very large shift in tailwater rating is shown in Figure 11-6. This figure compares the tailwater ratings for the natural conditions before construction of the Aliceville Lock and Dam on the Tennessee–Tombigbee Waterway with project conditions after construction was complete. The drastic shift of the rating is largely due to excavation of the downstream navigation channel which caused not only an increase in channel flow capacity, but also a significant decrease in channel roughness. The variation in a tailwater rating curve may shift toward more flow capacity, less flow capacity, or oscillate from one to the other and back again. The shift in rating may be abrupt, gradual, or sporadic. It may be caused by sediment erosion or aggradation, excavation or deposition of channel bed or bank material, variations in hydrologic events, loops in rating curves as flow transitions from the rising to falling flood stages, inaccurate estimates of channel roughness, or by man-induced events. The hydraulic engineer should ensure that project features are designed for the proper conditions. For example, for projects with loop rating curves, rising stages should be used for design of stilling basins and erosion protection and falling stages used for setting wall heights. Use of an average tailwater rating curve in this case may yield inadequate design for both wall height and the high-velocity flow areas. The designer might also perform a sensitivity study of various channel n values to ensure that an incorrect assumption does not lead to an inadequate design. It will be the primary responsibility of the hydraulic design engineer to recognize the potential for shifts in tailwater ratings, evaluate the magnitude and consequences of a shift, and communicate this knowledge to others on the design team.

11.8 OVERFLOW EMBANKMENTS.

11.8.1 General

Required length of overflow embankments is often determined by selecting the combination of number of gates, length of overflow section, flowage easement, and levee raising that has the least total cost. An example of an optimization study accomplishing this is given in Chapter 15. When the overflow section operates under only highly submerged conditions the shape of the crest is of little significance on capacity. Overflow sections having significant head differentials will require properly shaped crests (normally ogee), energy dissipation structures, and downstream channel protection. The relatively low embankment sections used on the Arkansas River were designed for submerged conditions with head differentials of up to

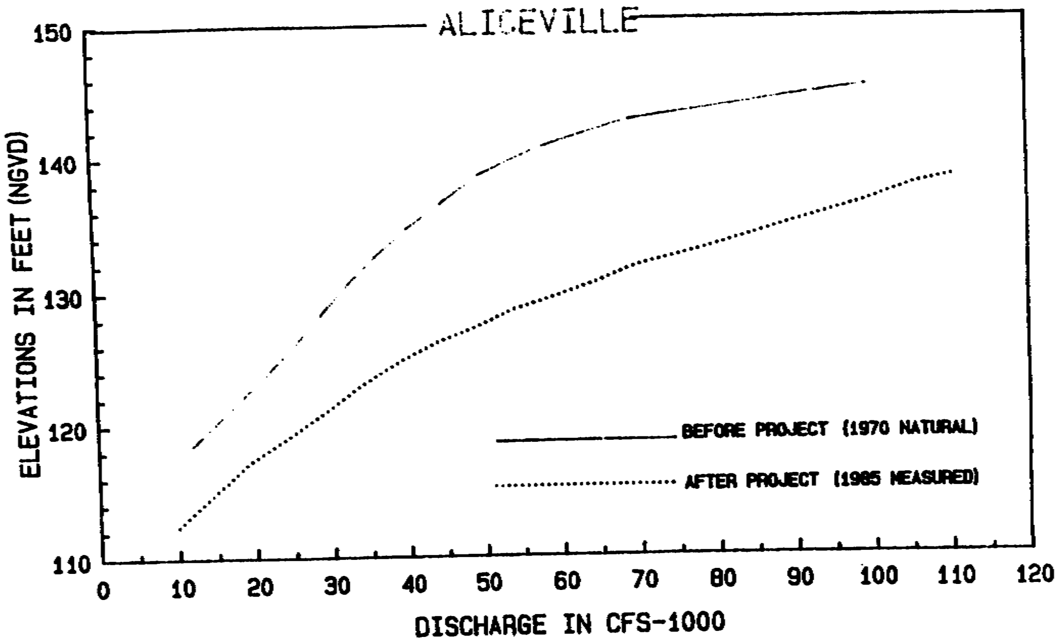


FIGURE 11-6. *Tailwater Rating Curves, Aliceville Lock and Dam, Tennessee–Tombigbee Waterway.*

3 ft. These riprap-protected embankments are either access or nonaccess embankments having trapezoidal cross sections with a 1*V*-on-3*H* upstream face and a 1*V*-on-4*H* downstream face. The access embankments have a paved roadway on the crown of the embankment.

11.8.2 Discharge over Uncontrolled Sections

The second type of uncontrolled overflow section is the concrete wall having considerable height and designed to operate under submerged conditions. Discharge coefficients for a rectangular cross-section and freeflow conditions are shown in EM 1110-2-1605 and HDC.

11.9 STILLING BASIN DESIGN

11.9.1 General

The purpose of the stilling basin is to reduce the kinetic energy of the flow entering the downstream exit channel. The stilling basin in conjunction with the downstream riprap ensures that local scour downstream of the structure will not undermine or otherwise threaten the integrity of the structure. Model tests can be used to find the optimum combination of stilling basin and downstream channel protection.

11.9.2 Influence of Operating Schedules

Operating schedules, both normal and emergency, are vital considerations in stilling basin design. Normal operating schedules should result in approximately equal distribution of flow across the outlet channel. Thus changes in the position of individual gates should be made in small increments with no two gate openings varying more than one foot. However, unusual or emergency operation must be considered. Unusual operation would include passage of floating debris (ice, logs, trash, etc.) through the gated structure during periods of minimum flow in the river. Such debris usually will begin to be drawn under a gate that is about one-third opened. Emergency operation would include design for one gate fully opened during periods of minimum flow which generally means minimum tailwater. Thus these operation requirements dictate a stilling basin that will adequately dissipate the excess kinetic energy at a low tailwater elevation.

11.9.3 Requirements for New Project Design

The following conditions are used to optimize stilling basin length and downstream scour protection thickness, size, and length. Structure foundation should be considered in determining the design condition. Structures founded on rock may have less restrictive energy dissipation and downstream protection requirements.

1. Uniform discharge through all spillway gates for a range of headwaters and tailwaters expected during project life.
2. Single gate fully opened with normal headwater and minimum tailwater. This condition would assume gate misoperation or marine accident. Minor damage to the downstream scour protection may occur as long as the integrity of the structure is not jeopardized. Single gate fully opened with above normal pool (perhaps the 50 to 100 year pool) should also be given consideration. This condition would simulate loose barges that could block several gates causing above normal pools as occurred at Arkansas River Lock and Dam No. 2 during December, 1982.
3. Single gate opened sufficiently wide to pass floating ice or drift at normal headwater and minimum tailwater. During preliminary design, a gate half opened can be assured to approximate ice- or drift-passing condition. Final design usually requires model studies to determine the proper gate opening. No damage should occur for this condition. For most low-head navigation structures, Conditions (2) and (3) result in free flow over the crest. The stilling basin design guidance presented in this chapter is for free flow. Stilling basins designed for submerged flow normally require a model study.

11.9.4 Hydraulics of Stilling Basins

Computations for d_1 and V_1 can be based on the assumption that there is no energy loss between the upper pool and the toe of the jump. The energy equation can be used to determine the entering depth and velocity into the stilling basin according to

$$\text{Upper Pool } \textit{Elevation} + \text{Velocity } \textit{Head Upstream}$$
$$= \text{Stilling Basin } \textit{Floor Elevation} + \frac{V_1^2}{2g} + d_1 \qquad \text{(11-8)}$$

Knowing the upper pool elevation, velocity head upstream (if significant), and discharge, V_1 and d_1 can be solved by trial and error for an *assumed* stilling basin floor elevation. Next the Froude number of the flow entering the stilling basin is computed according to

$$F_1 = \frac{V_1}{\sqrt{g d_1}} \qquad \text{(11-9)}$$

Then the momentum equation is used to determine the ratio between the depths before and after the hydraulic jump according to

$$\frac{d_2}{d_1} = 0.5(\sqrt{1 + 8F_1^2} - 1) \tag{11-10}$$

(This form of the momentum equation ignores the forces on baffle blocks in the analysis.) At this point, the assumed stilling basin elevation is checked against the available tailwater according to

$$\text{Tailwater Elevation for Given Discharge} \\ - \text{Assumed Stilling Basin Floor Elevation} = \text{Factor} \times d_2 \tag{11-11}$$

A new stilling basin floor elevation is assumed until Equation 11-11 is satisfied. Early stilling basin design guidance used a factor equal to 1.0. Recent guidance has allowed higher stilling basin floor elevations by setting this factor equal to 0.85 when used with baffle blocks and an end sill. The higher stilling basin floor elevation often improves performance at intermediate discharges and results in lower cost. Use of a factor less than 1.0 in Equation 11-11 can only be used in conjunction with Equation 11-10, the simplified momentum approach.

11.9.5 Recommendations from Results of Previous Model Tests

11.9.5.1 General. Model tests have been conducted at WES, Vicksburg, MS during which stilling basin designs were developed for one gate half or fully opened. The energy dissipators for one gate half or fully opened are not hydraulic-jump type stilling basins. These basins often have entering Froude numbers less than 4.0, which means they are inefficient and unstable and the flow will oscillate between the bottom and water surface resulting in irregular wave formation propagating downstream. Baffles and end sills help to stabilize low Froude number basins. Primary dissipation results from the impact of the jet against the baffles, which also assists lateral spreading of the jet, with tailwater as a supporting element. In a hydraulic-jump type stilling basin, tailwater is a primary force and baffles are supporting elements; lateral spreading of the jet, outside the confining walls, usually is not a consideration.

11.9.5.2 Basin Elevation. In a baffle-assisted hydraulic-jump type stilling basin, the apron must be placed at an elevation that allows tailwater to provide a depth on the apron of at least $0.85d_2$ (factor = 0.85). In the stilling basin considered herein, this has not proved to be a rigid requirement. However, for initial design of a specific project and until it has been established in model tests that conditions at that project will permit an apron at a higher elevation, it is suggested that the apron be placed at an elevation that will provide a tailwater depth of at least $0.85d_d$ for both one gate half or fully opened.

11.9.5.3 Basin Length. Model test results suggest a required length L_2 from toe of jump to beginning of 1*V*-on-5*H* up slope of

$$L_2 = 2d_1F_1^{1.5} \tag{11-12}$$

11.9.5.4 Baffles. The position and height of the first row of baffles have a major influence on stilling action. Baffle height and position recommended for the basins developed in numerous model studies are shown in Table 11.1. These basins designed for a single gate half or fully opened require higher baffle blocks than hydraulic-jump type basins. A second row of baffles is not required for maintaining the jump within the basin but is recommended to reduce attack on the downstream channel protection. These baffles should be the same height as those in the first row, placed with their upstream faces about two baffle heights downstream from the upstream faces of the first row and staggered with respect to the baffles in the first row. In cases where foundation requirements dictate a deep basin ($> d_2$), baffle blocks may not be required.

11.9.5.5 Gate Pier Extensions. Gate pier extensions are required to extend into the basin to a position five feet upstream of the baffles to prevent return flow from inoperative bays. The pier extension can be extended farther downstream if required for stability. These extensions are required to ensure adequate stilling basin performance for the single gate half and fully opened criteria given in Sections 11.9.3(2) and 11.9.3(3), respectively. The pier extensions should be at least one foot higher than the tailwater used for the single gate half or fully opened criteria. Pier extension width can be less than the main spillway piers.

11.9.5.6 End Sill. An end sill slope of 1*V* on 5*H* was effective in spreading the flow for single gate operation. The higher the end sill, the more effective it will be in spreading the jet during single gate operation, but there are limi-

TABLE 11-1.

Gate Opening	Height, h_b	Distance to First Row, L_1
Full	$0.25d_2$	$1.3d_2$
Half	$0.3d_2$	$1.5d_2$

tations. The higher end sill results in shallower depths in the exit channel and possibly higher velocities over the riprap. Of course, the top of the end sill should not be appreciably above the exit channel. Also, the end sill should not be so high that it causes flow to drop through critical depth and form a secondary jump downstream. To prevent this, the Froude number $F = V/\sqrt{gd}$ at the top of the end sill, calculated as described in the following section, should not exceed 0.86 for single gate guidance given in Section 11.9.3. In this calculation, V is difficult to determine because of spreading of the flow for single gate operation. A reasonable estimate for V is 80% of the velocity over the end sill without spreading based on bay width, discharge, and depth over end sill. The terms d and g represent depth of tailwater over the end sill and the acceleration due to gravity, respectively. Experiments in a rectangular channel indicate that tranquil flow becomes unstable when F exceeds 0.86; thus this limiting value. Excessive spreading will cause attack of boundaries in outside bays. Based on numerous model studies, the end-sill height varies considerably for basins designed for either fully or half-opened gate criteria. A value of 0.15 to $0.20d_2$ is recommended for basins designed for either a fully or half-opened gate.

11.9.5.7 Training Walls. The elevation of the top of the training walls is normally selected to prevent overtopping at all but the highest discharges. This is not a strict requirement for low-head navigation dams and training wall tops have been placed as low as two feet above the downstream normal pool elevation. This reduction in height should be model tested. Training walls are normally extended at a constant top elevation to the end of the stilling basin as shown in Figure 11-7(a). This, too, is not a strict requirement. The Red River design is shown in Figure 11-7(b). Adjacent project features and topography have a significant impact on training wall design.

11.9.5.8 Abrasion. Abrasion of concrete can be caused by the presence of gravel or other hard particles. Rock, gravel, scrap metal, and other hard material may find their way into the energy dissipator by various means. Rock may be carried into a stilling basin over the top of low monoliths during construction, by rollers or eddies bringing debris in from downstream, or by cobbles moving as bed load. Protection stone in the vicinity of the end sill should not contain stone sizes that can be transported by underrollers into the stilling basin. In some cases, the contractor may fail to clean out all hard, loose material after construction. During operation, rocks may be thrown in from the sidewall by the public, or fishermen using rocks for anchors may leave them behind. The elimination of such material may require specification of construction practices or proper restriction of the public during operation. In cases where it is believed that rock and gravel are being transported into the basin by rollers, all gates should discharge an equal amount of water.

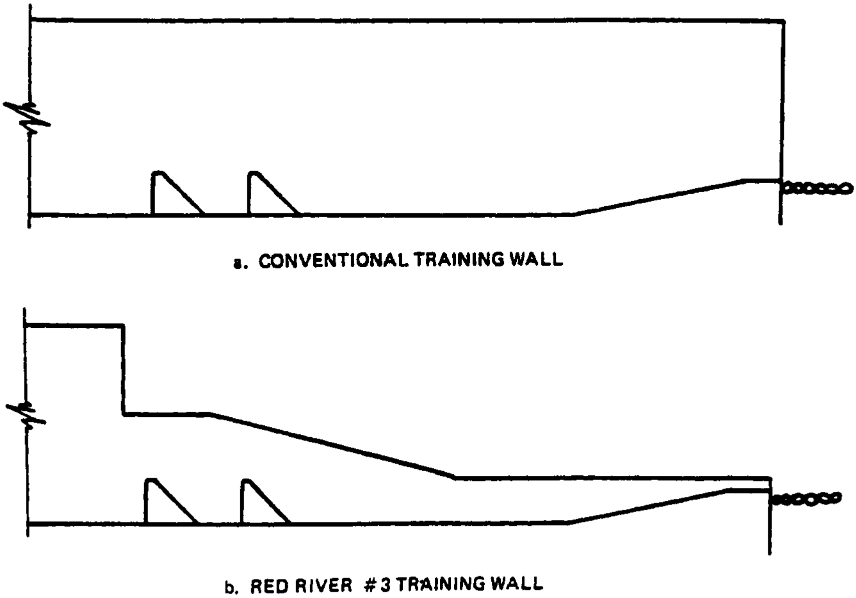


FIGURE 11-7. *Training Walls: (a) Conventional Training Wall; (b) Red River #3 Training Wall.*

11.9.5.9 Cavitation. Cavitation is the successive formation and collapse of vapor pockets in low-pressure areas associated with high-velocity flow. Cavitation damage can occur on the sides of baffle blocks, on the floor of a stilling basin just downstream from baffle blocks, and at construction joints near the upstream end of the stilling basin. Any surface discontinuity of the boundary into or away from high-velocity flow can cause cavitation. Relative movement of two concrete monoliths or slabs with a lateral construction joint so that the downstream slab comes to rest higher than the upstream slab produces a situation where cavitation may result. In any case where high-velocity flow tends to separate from the solid boundary, cavitation may be expected to exist. Cavitation is not normally a problem at low-head navigation dams because of the relatively low velocities. There is reason to believe that both abrasion and cavitation are responsible for damage at some structures. If a sizable depression in the concrete surface is eroded by abrasion, cavitation may then form and augment the damage. Likewise, abrasion can mask cavitation where both are occurring. In general, concrete damaged by cavitation has a ragged angular appearance as though material had been broken out of the mass. In contrast, damage caused by abrasion has a smoother or rounded appearance, such as would be caused by grinding.

11.10 APPROACH AREA

11.10.1 Configuration

The approach to the spillway should be greater than 3 ft below the crest of the spillway. An approach depth of 5 ft is recommended because most discharge calibration data were taken with this depth. Approaches with depths less than 3 ft can result in greater tendency for movement of the riprap in front of the structure for a single gate fully opened. Approaches having a deep trench in front of the structure can result in instabilities of the flow over the crest and may simply fill with sediment. The approach should be horizontal for a minimum of 50 ft and then sloped to the streambed at a rate not to exceed 1*V* on 20*H*.

11.10.2 Upstream Channel Protection

To prevent scour upstream of the structure, protection is required, particularly for single gate operation. An estimate of the required riprap size upstream of a navigation dam can be obtained by determining the approach velocity by taking the unit discharge (discharge/width of bay) and dividing by the depth (difference in elevation between the upper pool and the approach channel to the spillway). This provides an average velocity and depth that can be used in the following relation to determine the stone size required.

$$V = 1.2\left[2g\left(\frac{\gamma_s-\gamma_w}{\gamma_w}\right)\right]^{1/2} D_{50}(min)^{1/2} \tag{11-13}$$

This equation is the Isbash low turbulence relation given in HDC 712-1.

Placement underwater requires an increase in thickness of 50%. Single gate operation will generally be the most severe with respect to design of upstream riprap but hinged pool operation (as described in Section 18.1.3.3) should be evaluated. Concrete aprons have been used in place of riprap when riprap size becomes excessive. The riprap or concrete apron should be extended upstream a minimum distance equal to the head on the crest. If protection must be provided for the effects of sunken barges in front of the structure, the concrete apron should be used.

11.11 EXIT AREA

11.11.1 Configuration

For the condition of only a single gate discharging, configuration of the exit area has a major influence on stilling action. Abrupt side contractions and areas of unequal elevation across the channel cause side eddies to be intensified and thus hamper jet spreading. There is little agreement on the effectiveness of a preformed scour hole. Many projects have been designed with a deepened area downstream to lessen attack on the riprap. A relatively small amount of expansion, preferably both vertically and horizontally, will reduce the severity of attack of the channel boundary. However, there is a tendency for this deepened exit channel to exhibit stronger side eddies which tends to reduce spreading for single gate operation and can lead to a decrease in riprap stability. Final riprap configurations downstream from spillways should be model-tested and adjusted as necessary to ensure the adequacy of the protection. Based on the preceding field and model experiences the following guides for preliminary layout are suggested. Begin the riprap with the top of the blanket 1 to 2 ft below the top of the basin end sill. If possible, extend the riprap section horizontally. Where the streambed is higher than the end sill, slope the riprap upward on a 1V-on-20H slope. Where locks or other structures do not abut the spillway, the riprap section is extended up the bank-line slope. The toe of this slope should be set back 5 to 10 ft from the face of the spillway training wall.

11.11.2 Downstream Channel Protection

The size and extent of the riprap required in the exit area depend upon the effectiveness of the stilling basin, tailwater depth in the exit, and configuration of the exit area. The size of riprap required is almost always governed by either the fully or half-opened gate criteria or diversion conditions. As flow leaves the single gate bay, spreading occurs and the average velocity decreases in the downstream direction. The average velocity over the end sill can be 75 to 90% of the velocity without spreading. Use of 80% of the velocity over the riprap without spreading in the relation

$$V = 1.12\left[2g\left(\frac{\gamma_s-\gamma_w}{\gamma_w}\right)\right]^{1/2} D_{50}(min)^{1/2} \tag{11-14}$$

provides riprap size for use immediately downstream of the end sill. This equation is restricted to basins designed using the guidance presented in this chapter. This equation is the same form as the Isbash relation given in HDC 712-1. A comparison of the results given in model tests and Equation 11-14 is given in Table 11.2.

The large differences between model and computed results are largely due to difference in stilling basin performance, particularly the effects of a wide variation in end-sill height. These values should be used in preliminary design and verified in a physical model. Riprap gradations are given in EM 1110-2-1605. Thickness for placement in the dry should be $1.5D_{100}$(MAX) or $2.0D_{50}$(MAX), whichever is greater. Thickness for placement underwater

TABLE 11-2.

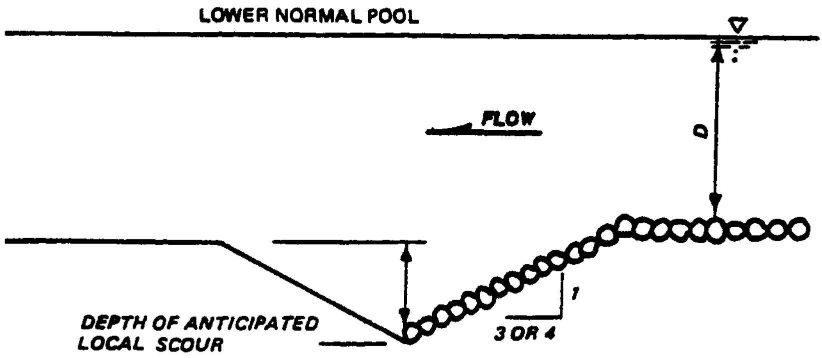
Project Name	Basin No.	Velocity over Riprap Without Spreading (ft/sec)	D_{50} Model (ft)	D_{50} Computed (ft)
L&D 26	16	29.9	3.8	4.3
Aliceville	6	19.4	2.2	1.8
Columbus	5	17.6	2.0	1.5
RR 1	16	30.2	1.5	4.4
RR 2	13	31.1	2.5	4.7
RR3	2	25.8	2.6	3.2
L&D 26	30	14.7	2.6	1.0
Columbus	4	13.4	1.7	0.9
RR 1	17	16.8	1.5	1.4
RR 2	7	23.4	1.1	2.7

should be increased 50%. The top of the riprap should be placed 1 to 2 ft below the top of the end sill. Total length of riprap protection on the channel invert downstream of the end sill ranged from $4d_2$ to $27d_2$ in numerous model studies. A minimum length of $10d_2$ downstream of the end sill is recommended for fully or half-opened gate design. The change in riprap size in the downstream direction should be as shown in Table 11.3.

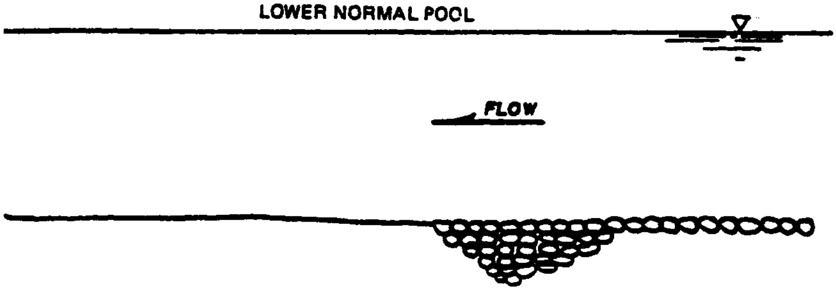
Riprap creates locally high boundary turbulence that leads to local scour at the downstream end of the riprap blanket. This requires that the downstream end of the riprap be "keyed in" as shown in Figure 11-8. Method A requires extending the riprap to a depth equal to or greater than the anticipated scour. Method B provides sufficient riprap in a trench to launch as local scour occurs. The need to "key in" the riprap is most apparent at projects where the downstream riprap protection does not extend $10d_2$ below the end sill. In some cases, adjacent vertical walls inhibit spreading of the jet during single gate operation and increase the size of riprap required. In cases where the riprap size becomes excessive, concrete aprons or

TABLE 11-3.

Distance	Riprap Size
$3d_2$	x = thickness immediately downstream of end sill
Next $3d_2$	$0.8x$
Next $2d_2$	$0.6x$
Next $2d_2$	$0.4x$



a. METHOD A - EXTEND TO ANTICIPATED SCOUR DEPTH



b. METHOD B - PROVIDE ROCK-FILLED TRENCH

FIGURE 11-8. Methods for Transitioning from Riprap to the Unprotected Downstream Channel: (a) Method A—Extend to Anticipated Scour Depth; (b) Method B—Provide Rock-Filled Trench.

grout-filled bags have been used. Side-slope riprap is normally the same size as the invert. Granular filters are recommended for riprap placement adjacent to structures.

11.12 SPILLWAY GATES

Various types of gates have been used as control devices at Corps of Engineers' navigation projects. Examples are tainter gates, roller gates, vertical-lift gates, and the like. The current most commonly used and recommended control is the tainter gate.

11.13 Gate Types and Selection

The types of gates used at Corps of Engineers' navigation dams and factors considered in the selection of type of gate at a specific project are described in the following paragraphs.

11.13.1 Roller Gates

A roller gate is a long metal cylinder with "ring gears" at each end that mesh with inclined metal racks supported by the piers. The cylinder is braced internally to act as a beam to transmit the water load into the piers. The effective damming height of the structural cylinder can be increased by means of a projecting apron that rotates into contact with the sill as the gate rolls down the inclined racks. The gate is raised and lowered by means of a chain wrapped around one end of the cylinder and operated by a hoist permanently mounted in the pier. The rolling movement of the gate and the limited amount of frictional contact at the sealing points permit comparatively fast operation with a small expenditure of power. Roller gates have been built with a damming height of 30 ft, with lengths up to 125 ft on pile foundations and 150 ft on rock foundations.

11.13.2 Tainter Gates

A tainter gate in its simplest form is a segment of a cylinder mounted on radial arms that rotate on trunnions embedded in the piers. The tainter gate is considered the most economical, and usually the most suitable, type of gate for controlled spillways because of its simplicity, light weight, and low hoist-capacity requirements. The use of side seals eliminates the need for gate slots that are conducive to local low-pressure areas and possible cavitation damage. The damming surface consists of a skin plate and a series of beams that transmit the water load into the radial supporting arms. The tainter gate is raised and lowered by chains or wire rope attached at both ends, since the tainter type is less capable of resisting torsional stress than the roller gate. Gates may be manipulated by a traveling hoist, or by individual hoists, depending upon the desired speed of operation and consideration of costs. Tainter gates require more power for operation than roller gates of similar size, since nearly all the weight of the gate is suspended from the hoisting chains whereas the weight of a roller gate is about equally divided between the chain and the pier. Counterweights will reduce the power required, but will add to the total weight of the structure. Tainter gates built to heights of 75 ft and lengths of 110 ft have been used for navigation dams. It is desirable but not mandatory that the trunnions of tainter gates be placed above high water, and essential that the gate itself be capable of being raised above high water. Cox (1972), identifies three types of tainter gate mounting arrangements and describes, with pertinent geometrical data, the gate design and mounting arrangement at 176 Corps of Engineers' projects.

11.13.3 Vertical-Lift Gates

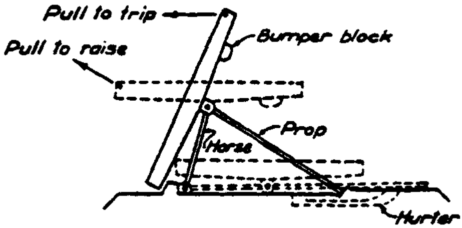
The vertical-lift gate moves vertically in slots formed in the piers and consists of a skin plate and horizontal girders that transmit the water load into the piers. For the larger heads, the gate must be mounted on rollers to permit movement under water load. The vertical-lift gate, like the tainter gate, must be hoisted at both ends, and the entire weight is suspended from the hoisting chains. Piers must be extended to a considerable height above high water in order to provide guide slots for the gate in the fully raised position. Vertical-lift gates have been designed for spans in excess of 100 ft. High vertical-lift gates are sometimes split into two or more sections in order to reduce hoist capacity, reduce damage to fingerlings passing downstream, or ease passing ice and debris. However, this does increase operating difficulties, because the top leaf or leaves have to be removed and placed in another gate slot.

11.13.4 Other Types

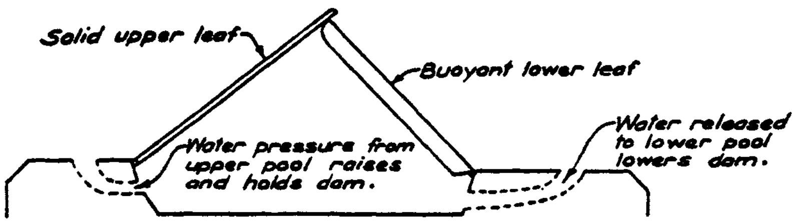
Various other types of damming surfaces have been used for navigation dams. These usually have been relatively slow-acting adaptations of stop-log bulkheads or needle dams for operation by hand or limited amounts of mechanical power. The stop-log type of dam consists of piers with vertical slots in which timbers or built-up sections of skin plate and girders are stacked to the desired height. The needle dam consists of a sill and piers that support a girder designed for horizontal loading. Needles or shutters of comparatively narrow width are placed vertically or inclined downstream to rest against the girder and sill and are held in place by the water load. Other navigation dam types such as wicket (Chanoine and Bebout), bear trap, and Boulé dam (see Figure 11-9) are movable dams.

11.13.5 Selection of Gates

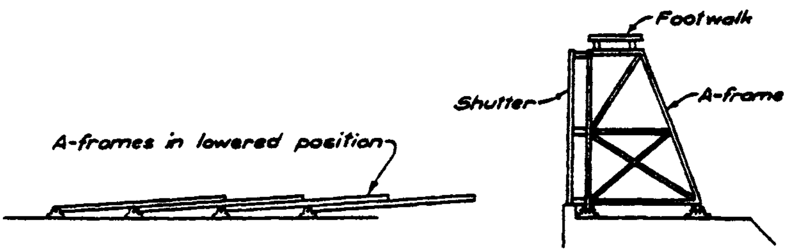
Gates that best meet the operational requirements of the proposed spillway should be provided. Where two or more types of gates appear equally efficient, from a functional standpoint, the decision should be made upon an economic basis. Tainter gates have been used in most recently constructed navigation dams. Advantages that may be ascribed to tainter gate installations are:



WOODEN CHANOINE WICKET



BEAR TRAP DAM



BOULE' DAM

FIGURE 11-9. *Typical Movable Dams.*

1. lighter lifting weight with smaller hoist requirements;
2. adaptability to fixed individual hoists and push-button operation. Individual hoists may have a lower first cost than gantry cranes and require fewer operating personnel;
3. less time required for gate operation (more than one gate can be operated at the same time;
4. favorable discharge characteristics.

Disadvantages of tainter gate installations are:

1. encroachment of radial arm on the water passage;
2. the necessity for excessively long radial arms where the flood level, to be cleared, is extremely high.

The advantages of a vertical-lift gate installation are:

1. provision of a clear gate opening with no encroachment, when raised, of any part of the gate structure on the water passage;
2. more adaptable to extreme pool fluctuations in that it is lifted bodily out of the water.

Some of the disadvantages encountered in the use of vertical-lift gates are:

1. heavier lifting load which requires greater hoist capacity and often necessitates a "split-gate." The split-gate increases operation difficulties;
2. not favorable for adaptation to fixed individual hoist operation. The most common method of operation is by gantry crane which may have a greater first cost than do fixed hoists and also requires more operating personnel;
3. greater time required for gate operation because normally only one crane is provided. Time element may be especially significant at sites subject to flash floods;
4. gate slots lead to potential cavitation and debris collection.

11.14 TAINTER GATE DESIGN

Design guidance is given in the following paragraphs.

11.14.1 Gate Seal Design and Vibration

Many laboratory and field studies have been done with instabilities (gate vibration and oscillation) at Corps of Engineers' projects. The following guidance is recommended for gate seal design.

1. The configuration of the tainter gate lip and bottom seal is a major factor in setting up flow conditions that cause gate vibrations. Ideally, tainter gate lips should provide as sharp and clean a flow breakout point as possible. Supporting structural members downstream from the lip should be kept as high and narrow as possible.
2. Rubber seals should not be used on the gate bottom unless water conservation requirements cannot tolerate the normal leakage. If required, a narrow rubber bar seal may be attached rigidly to the back side of the gate lip. However, even minor variations from this seal design can result in vibrations. Consideration should also be given to providing a rubber seal in the gate-sill bearing plate. However, such seals are normally more difficult to maintain than gate-mounted seals.
3. In wider tainter gates with high trunnion anchorages, the hydrostatic force of the pool against the skin plate tends to bow up the lip at the center of the gate.
4. Structurally, the gate members should be rigidly designed to limit possible gate flexing under hydraulic loads. Rigid rib-to-girder welded connections and stiffener braces between the bottom girders and the cantilevered portion of the skin plate provided the necessary rigidity on the Arkansas gate designs.
5. Gate side seals should be designed with sufficient flexibility to remain in contact with the side seal plates at all gate openings and for all probable gap openings as might be caused by construction inaccuracies, gate skews, gate temperature shrinkage and expansion, and normal structural settlements. The side seals should initially be set with a slight deflection forcing the seal against the seal plate. Debris that becomes wedged between the seal and seal plate should be cleaned out at regular intervals.
6. Unusual gate designs or features should be tested in model facilities or, if practical, on existing spillway gates that have similar geometric and hydraulic conditions to ensure against cavitation tendencies.
7. No spillway tainter gate design or feature should be predicated, or made contingent, on the use of any specific gate operating scheme or plan.

11.14.2 Surging of Flow

Design criteria have been developed to prevent periodic surging of flow on spillway tainter gates. Model tests have indicated that the most effective means of eliminating the periodic surge on the tainter gates is to decrease the length of crest piers upstream from the gates or to increase the width of gate bays, or both. For low-overflow spillways, the gate-bay width should be equal to or greater than:

1. 1.1 times the maximum head on the weir crest for which the gates control the discharge when the length of crest piers is less than 0.3 times the gate-bay width;
2. 1.25 times the maximum head on the weir crest for which the gates control the discharge when the length of crest piers is between 0.3 and 0.4 times the gate-bay width. The maximum gate opening for which tainter gates will control the discharge should be taken as 0.625 times the head on the weir crest. By utilizing the spillway discharge curves for various gate openings, the maximum head on the weir crest for which the gates will control the discharge can be determined.

11.14.3 Gate Seat Location

The gate seat should be located at the beginning of the parabolic drop or within two feet upstream of that point for low-head navigation structures. This location will help the jet adhere to the downstream face of the crest.

11.14.4 Tainter Gate Trunnion Elevation

Trunnion elevation is set above most floods. Typical submergence allowed is a maximum of 5 to 10% of the time.

11.14.5 Top of Gates, Closed Position

When in the closed position, the gates should have at least one foot of freeboard above the normal upstream pool. On large pools where fetch for wave setup is large and water conservation is important, more than one foot may be required.

11.14.6 Bottom of Tainter Gates, Raised Position

Gates should be designed to clear the highest flood with allowance for floating debris. Typical clearance is one to five feet above the PMF. Special consideration may be appropriate for projects with major flood levees along the overbanks. Often the maximum stage will occur just before the levees are overtopped. Subsequent discharge increases would result in lowered stages because of levee failure and dispersion of flows through the protected areas. For spillways in such locations, the maximum gate-opening height would be set at one foot above the adjacent levee crown elevation. Another consideration is raising the bottom of the gates to allow accidental passage of barges through the gate bays without damage to the tainter gates.

11.14.7 Gate Radius

Skin plate radius ranges from 1.0 to 1.2 times the damming height of the gate. The radius of the gate is affected by the vertical distance between the bottom of the gate in the lowered position and low steel of the gate in the raised position. Spillway bridge clearance may also be a factor in determining the gate radius and the trunnion location.

11.14.8 Submersible Tainter Gates

Submergible tainter gates were developed to allow passage of ice without having to use large gate openings. Two types have evolved, the type in which the top of the gate can be lowered below the normal upper pool elevation and the piggyback gate. Both types are shown in Figure 11-3. A shaped lip on the top of the gate can be used to keep the flow off the back of the gate. A listing of projects having submergible tainter gates is given in EM 110-2-1605. Some of these projects have experienced scour and/or vibration problems. Lifting chain or cable loads are much greater in deep submerged positions and must be considered in machinery costs. At Lock 24, Upper Mississippi, submerged tainter gates have only been effective for passing light floating ice.

11.15 VERTICAL-LIFT GATE DESIGN

Reference is made to EM 1110-2-2701 and EM 1110-2-1603 for design of vertical-lift gates.

11.16 SPILLWAY PIERS

The hydraulic performance and discharge capacity of spillways are affected by the pier designs. The following factors need to be considered.

11.16.1 Thickness

Pier thickness is dependent upon structural requirements and is generally a function of the bay width and pier height. Pier widths for the spillways as determined by model tests vary from 8 to 15 ft.

11.16.2 Supplemental Closure Facilities

Bulkheads are provided on all gated navigation spillways to permit gate maintenance without draining the pool. Bulkhead slots are located in the piers and have their upstream side about one pier thickness downstream from the pier nose. The slots must be upstream far enough to ensure that the bulkheads will clear the gate-raising mechanisms while being placed. Occasionally, bulkhead slots are provided on the downstream ends of piers also. These bulkheads would permit dewatering and inspection of the spillway gate sill. When lower pool levels are higher than the gate sill, inspections must be made by divers if these bulkheads are not provided.

11.16.3 Pier Nose Shape

A semicircular pier nose shape is the most common and generally satisfactory design. An ogee shape (Type 3, HDC 111-5) was found to be only slightly more efficient than the semicircular shape. All the Arkansas River navigation spillways have a curved nose leading to a 90 degree point (similar to ogee). A structural angle is embedded in the point. The angle has helped to protect the piers from being damaged by colliding barges and other objects. This shape is very efficient when the gates on both sides of the pier are set at equal openings. However, when gate settings are very different, the sharp pier nose causes a flow separation from the pier on the larger gate-opening side causing a reduction in efficiency.

11.16.4 Barge Hitches

If floating plant is used for spillway or spillway gate maintenance, tie-up posts should be added to both the upstream and downstream ends of the piers. By recessing the posts back from the pier face, they will cause minimal flow disturbances.

11.17 ABUTMENTS

Long-radius abutments are used infrequently at low-head navigation dams because the spillway is normally located for straight approach flow which minimizes need for large abutments, and operation of adjacent locks, overflow sections, powerhouses, and the like, would be hindered by large abutments. Abutment radius used on many projects were the same as the interior piers that equaled one half of the pier width.

11.18 NAVIGABLE PASSES

Navigable passes permit the passage of tows over low-head dams without the requirement for locking. These may be appropriate at some dams if certain conditions obtain. These include stages high enough to permit open-river navigation for a significant portion of the year, individual high-water periods usually of considerable duration, and a gate regulating system commensurate with the rate of river rise and fall. The benefits of a navigable pass may include lower lock wall heights and lower tow operating costs when lockage is unnecessary. This may be offset by higher maintenance costs for locks that sustain relatively frequent overtopping. In addition to dams for which a navigable pass is included as an element in their configuration, many other dams have high-water navigation over a weir section. This includes both dams with gated and weir sections as well as dams entirely constructed as fixed-crest structures. These dams also may require less lock-wall height. The design of a navigable pass must provide for sufficient clear width for safe passage of tow traffic, including poorly aligned tows. At some locations this may include two-way traffic. In addition, the pass must have sufficient depth for tows of the authorized draft, including a buffer to account for overdraft, tow squat, and so on. Model studies have shown that a navigable pass should have a *minimum* cross-sectional area 2½ times the area blocked by a loaded tow. Current direction should be aligned normal to the axis of the navigable pass, and velocity through the pass must be low enough for upbound loaded tows of the horsepower range that operates on the waterway. A model study should be considered in the design of a navigable pass. At the present time, the Corps is operating dams with navigable passes on the Ohio and Ouachita Rivers. Pass widths vary from 200 ft on the Ouachita to 932 and 1,248 ft on the Ohio River. In addition, the Corps operates dams on the Illinois Waterway at which tows transit the regulating wicket section during higher stages. Gate types for navigable passes include Chanoine wickets (Figure 11-18) and hydraulically operated bottom-hinged gates. Fabridam has also been used but has experienced considerable problems with vandals and debris punctures. Drum gates are under consideration for a replacement structure on the Ohio River.

11.19 LOW-FLOW AND WATER QUALITY RELEASES

Provision for sluices as part of the main spillway or a separate outlet works to accomplish low-flow or multilevel releases should be designed according to EM 1110-2-1602.

11.20 FISH PASSAGE FACILITIES

Most fish passage facilities are located on the Columbia and Snake Rivers in the Pacific Northwest. Chapter 12 describes these fish passage facilities.

11.21 EXAMPLE DESIGN

An example design of a navigation dam and downstream scour protection can be found in EM 1110-2-1605.

11.22 SOURCE

The majority of information for this chapter came from US Army Corps of Engineers Manual EM 1110-2-1605, *Hydraulic Design of Navigation Dams*.

Chapter 12

OTHER FACTORS TO BE CONSIDERED

12.1 EFFECTS OF LOCKS AND DAMS ON SEDIMENT MOVEMENT

12.1.1 Spillway Operation

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

12.1.2 Hinged Pool Operation

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

12.1.3 Open-River Conditions

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

12.1.4 Depths in Upper Lock Approach

Although the lock and dam structures are used to maintain a minimum pool level some 20 ft or more above the natural low-water plane, depths in the upper lock approach cannot always be maintained without regulating structures (Figure 12-1). This is particularly true if the alignment of the channel is such that there is a natural tendency for shoaling along the lock side of the channel. Manipulation of the gate opening would generally be ineffective in removing the shoal since most of the sediment movement occurs during open-river conditions. The tendency for shoaling on the lock side of the channel can be determined by studying the channel configurations before the structure is built and investigating the tendency of the channel to cross toward the opposite side of the river.

12.2 HARBORS AND MOORING AREAS

12.2.1 Location

The development of commercial traffic on inland waterways will depend to a considerable extent on the availability of adequate mooring areas, fleet-

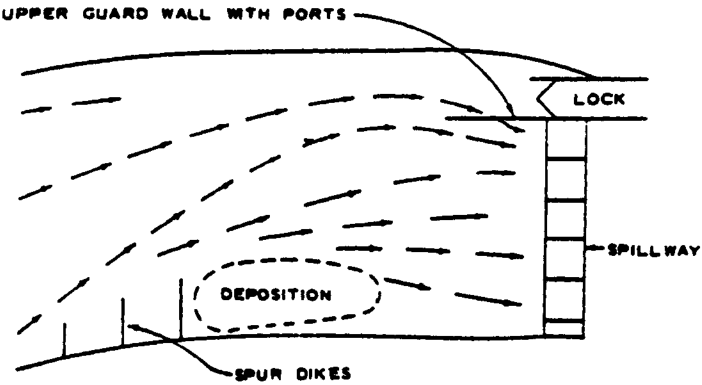


FIGURE 12-1. Training Structures Designed to Maintain Depths in the Upper Lock Approach.

ing areas, and docking and harbor facilities. In many cases, docking and harbor facilities are provided as part of the development or improvement of the waterway for navigation in cooperation with local interest. Unless these facilities are carefully planned, hazardous conditions could exist, particularly when placed along the bank adjacent to the main channel of the waterway. As a general rule, these facilities should not be placed close to lock approaches, on the concave side of a sharp bend, just upstream of a bridge, or where the channel tends to be of limited width.

12.2.2 Inland Harbors

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

12.2.3 Harbor Entrances

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

12.2.4 Effects of Currents

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

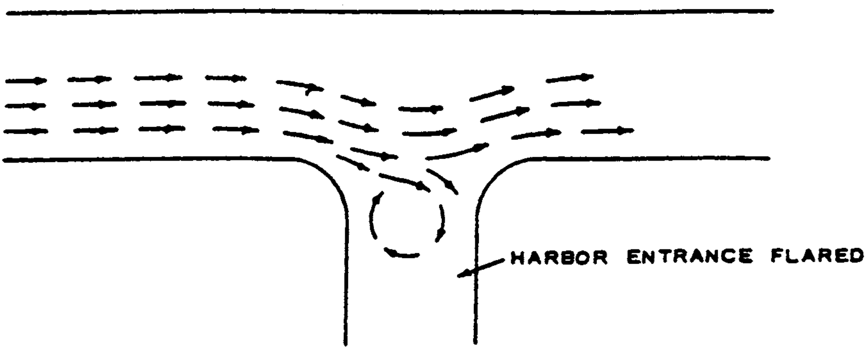


FIGURE 12-2. Currents with Flared Entrance.

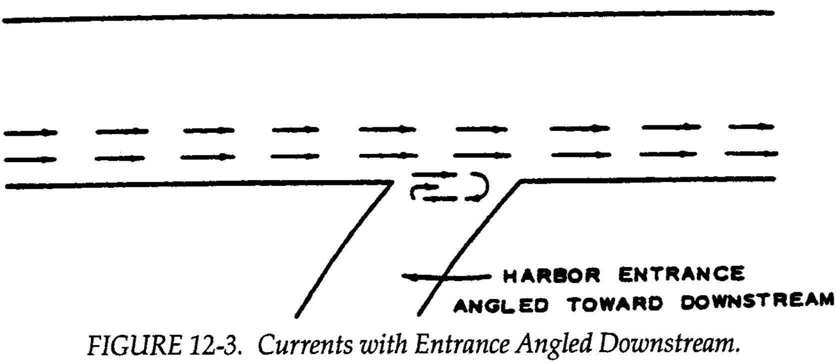


FIGURE 12-3. Currents with Entrance Angled Downstream.

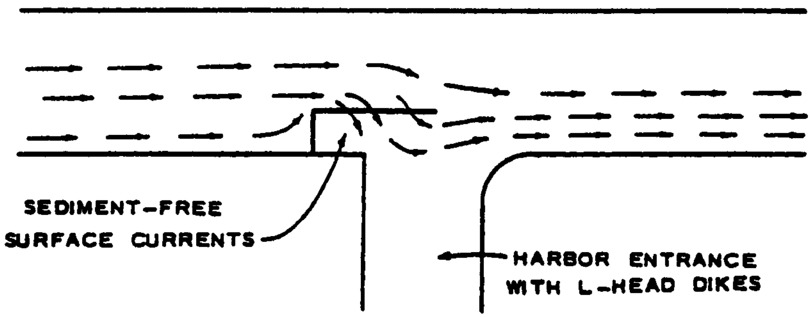


FIGURE 12-4. L-Head Dike Used to Reduce Shoaling in Harbor Entrance.

ing their direction (Figure 12-4). Another new concept to reduce shoaling in branched channels is a current deflector wall. This wall, shown in Figures 12-5 and 12-6, will break up the eddy current shown in Figure 12-2 and reduce the shoaling caused by the eddy current. Both model and prototype studies of this concept were successful at the Kohlfleet Harbor in Germany. The studies estimated a 40% decrease in the annual dredging requirement as a result of the current deflector wall construction.

12.2.5 Old Bendways

Development of harbors in old bendway channels bypassed by a cutoff will generally require the closure of one end of the channel (usually the upper end) and structures such as stepped-up dikes, L-head dikes, or wing dikes similar to those used in the lower lock approach to reduce or eliminate the tendency for shoaling in the entrance at the lower end of the bendway (Figure 8-1).

12.2.6 Harbor Design Guidance

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

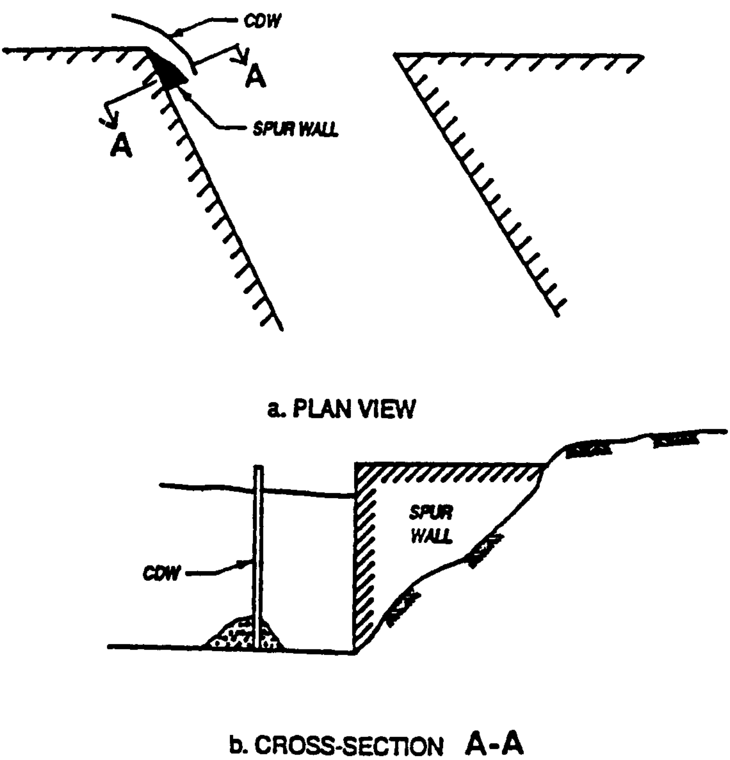


FIGURE 12-5. Current Deflector Wall (CDW).

endangering equipment moored in the harbor or harbor facilities and without excessive maneuvering. The size and shape of the turning basin should be a matter of judgment based on the size of tows using the harbor, type of commodity handled, traffic congestion anticipated, safety, and efficiency.

12.2.7 General

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

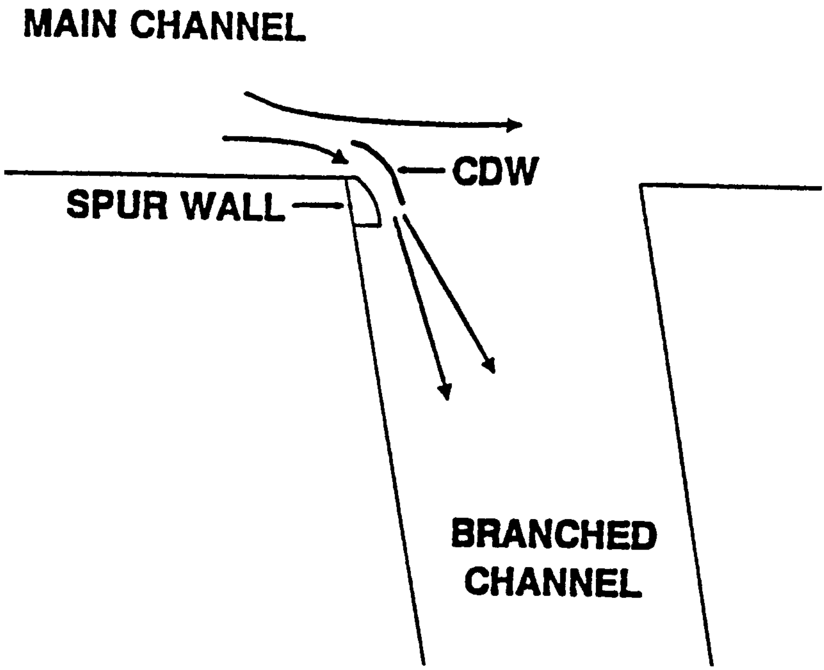


FIGURE 12-6. Current Deflector Wall Principle of Operation.

12.2.8 Modification of Locks

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

12.2.9 Lock Approaches

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

12.2.10 Lock Replacement or Addition

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

12.2.11 Modification of Channel Dimensions

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

12.2.12 Current Alignment

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

12.2.13 Bridges

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

12.3 SPECIAL DESIGN FEATURES

12.3.1 Special Features

Some special features that could have a significant effect on navigation conditions, operation and maintenance of the waterway, and/or cost of the project are discussed in the following sections.

12.3.2 Debris Control

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

12.3.3 Standardization

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

12.3.4 Emergency Closure

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

12.3.5 Impact Barriers

During the period 1968 to 1977 there were in excess of 350 reported collisions of barges with miter gates at the 260 Corps of Engineers Locks. Both repair costs and lost navigation benefits were considerable. One method of reducing the chance of these collisions is to provide impact barriers. Barriers should be provided at locks when a gate failure would cause loss of life or the repair cost and lost navigation benefits would justify the barrier cost. If barriers are considered necessary, they should be designed to withstand the impact of a full-size loaded tow traveling at a reasonable speed. Some of the provisions for the prevention of accidental damage to miter gates that should be considered are as follows.

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

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

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

12.3.6 Water Conservation

The competing interests for water can often provide a compelling case for conserving lockage water. Some water conservation measures for a navigation project with locks and dams could be the following.

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

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

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

12.3.7 Mooring Facilities

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

12.4 EFFECTS OF SURFACE WAVES

12.4.1 Waves Generated by Traffic

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

12.4.2 Wind Waves

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

12.4.3 Prototype Measurements

The size of traffic-created waves on the Ohio River and some of their effects on the environment were measured and evaluated by the US Army Corps of Engineers. These measurements covered a large number of vessels of different sizes moving upstream and downstream at different speeds and distances from the bank line. Preliminary analysis and evaluation of the results indicated the following general conclusions.

1. The size of waves approaching the bank increased with the size, draft, and speed of the tow and decreased with increase in the distance from the bank.
2. Waves created by small recreational vessels can be as large or larger than those created by towboats.
3. Wave sizes tended to be greater with greater depth (during high water) and tended to become smaller approaching a sloping bank or beach than when approaching a vertical bank.
4. The effects of traffic on the physical and biological components of the Ohio River were generally insignificant in comparison with ambient and natural changes. Changes produced by traffic were generally small and of short duration. The largest wave measured had a height of 3.3 ft produced by a towboat with 9 loaded barges moving downstream at a speed of about 12 mph and 200 ft from the water's edge. The measurement was made about 15 ft from the water's edge where

the depth was about 5.5 ft. It should be noted that the Ohio River is wider and deeper than most streams or inland waterways. Conditions in restricted waterways could be considerably different.

12.5 VESSEL TRAFFIC MANAGEMENT

12.5.1 General Concepts

Vessel traffic management encompasses a wide range of US Coast Guard activities including aids to navigation, vessel routing systems, regulated navigation areas (RNAs), navigation rules, voice communications, and vessel traffic services (VTS). Vessel traffic management attempts to establish two basic principles: good order and predictability. The objectives served by these principles include:

a. reduction in the rate of collisions, rammings, groundings, and ensuing environmental harm;
b. facilitation of vessel traffic movement;
c. provisions for all-weather navigation capability in certain areas; and
d. reduction in the rate of fire, explosion, and pollution casualties, and the probability of a port or waterway catastrophe.

12.5.2 Management Levels

Coast Guard Vessel Traffic Management exists on two levels: passive and active.

12.5.2.1 Passive Management. Passive traffic management is any form of traffic management where extent of compliance is vested solely with the user. The "rules of the road," aids to navigation, traffic separation schemes, and RNAs are all forms of passive management. Depending on the configuration of a particular port or waterway and complexity of its vessel traffic patterns, one or more passive management techniques may be established to achieve a desired level of safety and protection of the environment.

12.5.2.2 Active Management. Active traffic management involves direct interaction between the government and the user to ensure compliance with government requirements. Active management is used only in those areas where passive management techniques and procedures are inadequate to provide a desired level of safety and protection of the environment. When personnel not aboard a vessel become involved in the vessel's operation, vessel traffic management becomes active. VTS is the most common form of active traffic management. A VTS provides the person in charge of the vessel with information critical to safe navigation that would not normally be available without the VTS. Examples of such information include notifications of hazards to navigation, traffic advisories, and aids to navigation discrepancies. However, it remains the ultimate responsibility of the *master* to control the vessel movement to ensure the safe passage.

12.5.3 Federal Aids to Navigation

Aids to navigation fall into two categories; audio-visual and electronic. Audio-visual aids include:

a. buoys,
b. day marker,
c. fog signals,
d. light houses, and
e. beacons.

Figures 12-7 and 12-8 show types of aids. Figures 12-9 and 12-10 show aid placement. Electronic aids to navigation provide continuous, all weather position-fixing capability through the use of long-range aids to navigation (LORAN) and radio beacons (RACONs).

12.5.4 Privately Maintained Aids

Private aids to navigation are those that are legally required or personally desired by a property owner to be displayed. The aids could be on marine structures or private channels.

12.5.5 Navigation Rules

The regulations, formerly referred to as the "rules of the road," have the primary purpose of preventing collisions between vessels. The old Inland Western Rivers and Great Lakes Rules were combined into the new unified Inland Navigation Rules. Enforcement authority for these rules is vested in the Coast Guard. The Navigation Rules combined with aids to navigation, constitute the most basic form of traffic management. No vessel traffic management system relieves shipboard personnel from compliance with these Navigation Rules.

12.5.6 Voice Communications

The Vessel Bridge-to-Bridge Radio Telephone Act provides a positive means for operators of approaching vessels to communicate their intentions to one another. VHF-FM Channel 13 (156.65 mHz) has been designated as

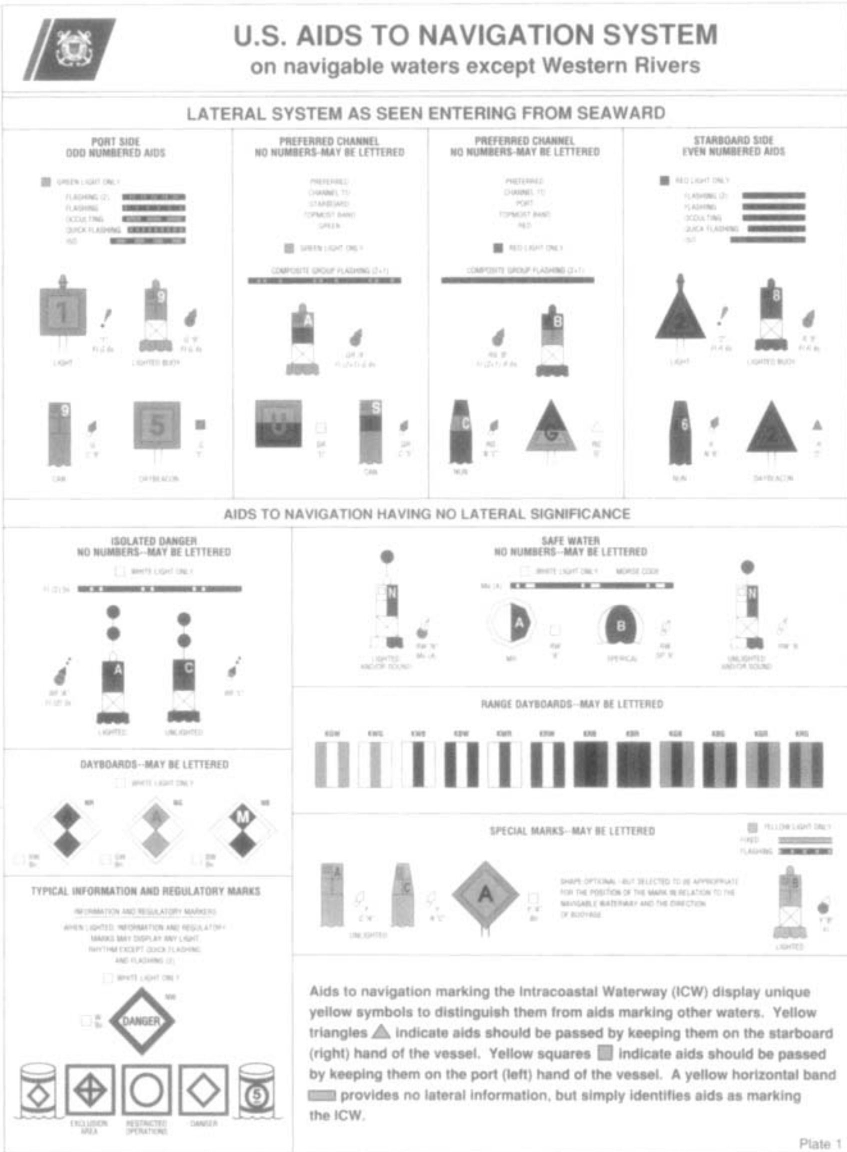


FIGURE 12-7. US Aids to Navigation System on Navigable Waters Except Western Rivers.

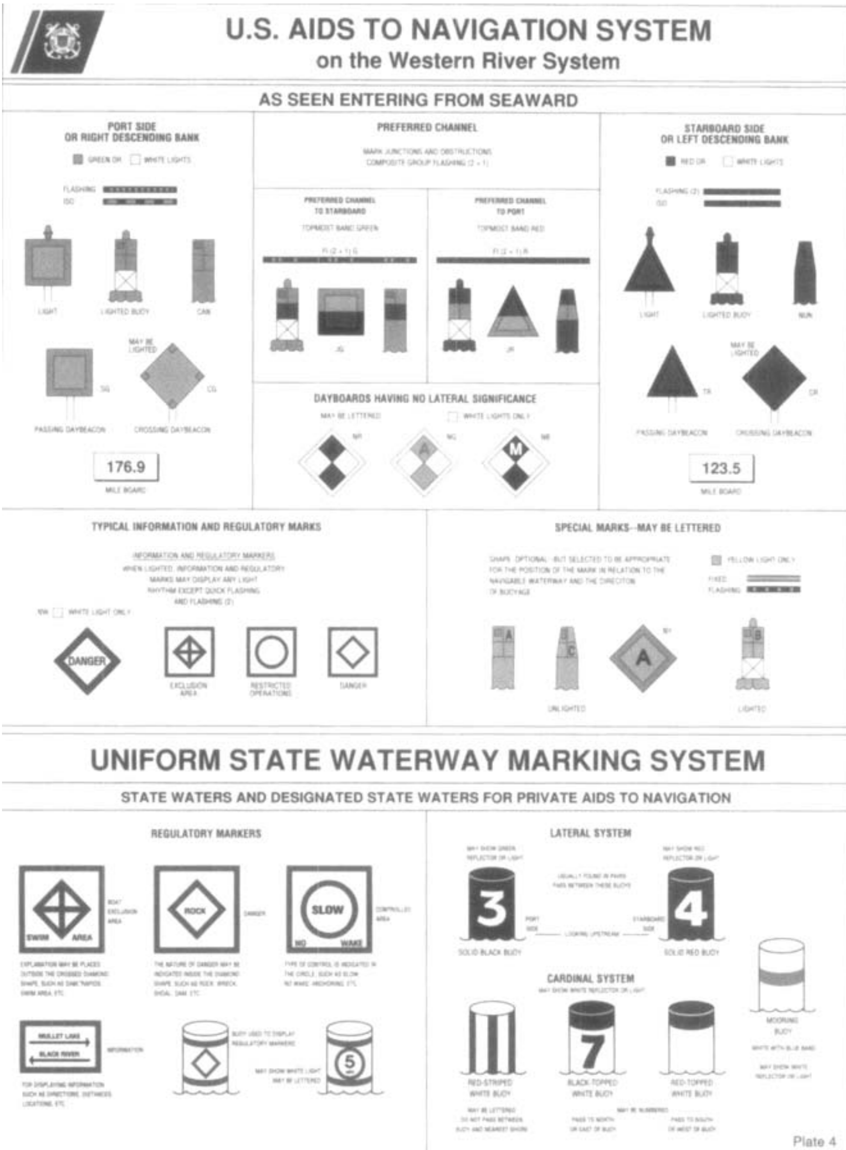


FIGURE 12-8. US Aids to Navigation System on Western Rivers.

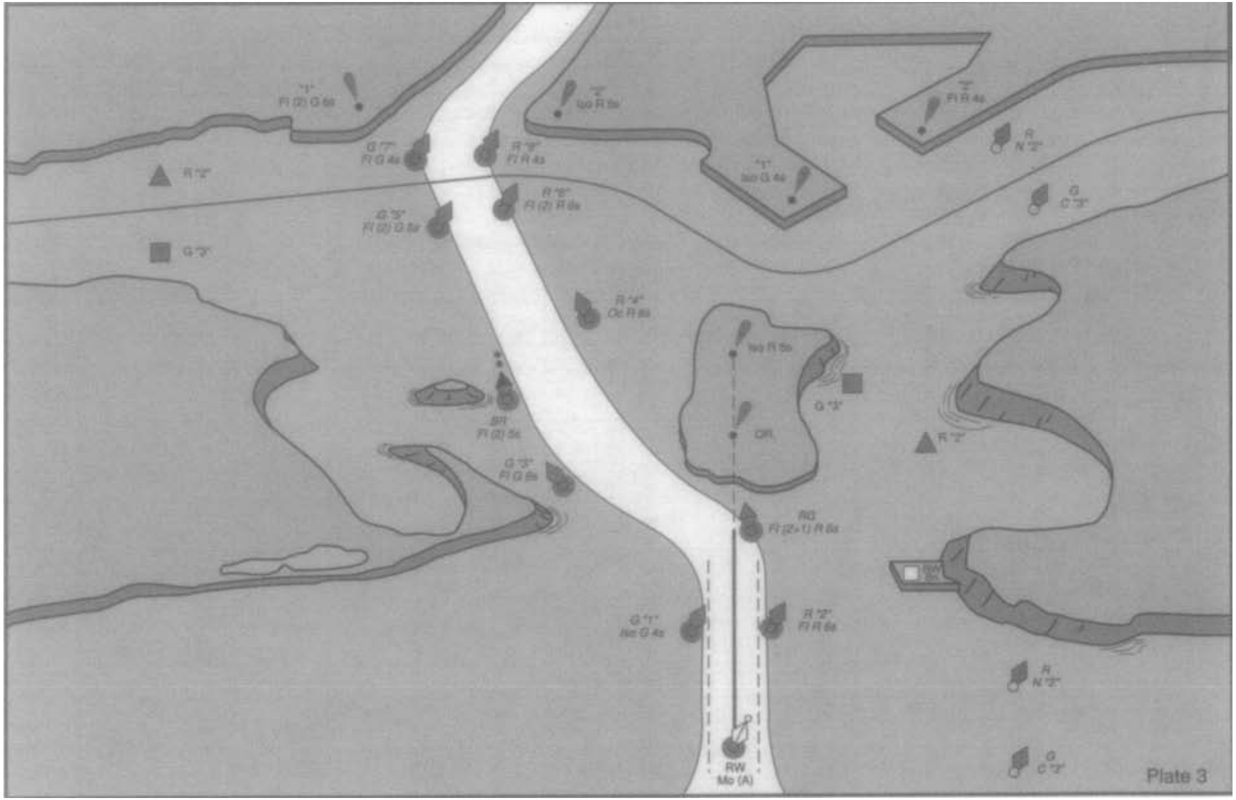


FIGURE 12-9. Fictitious Nautical Chart.

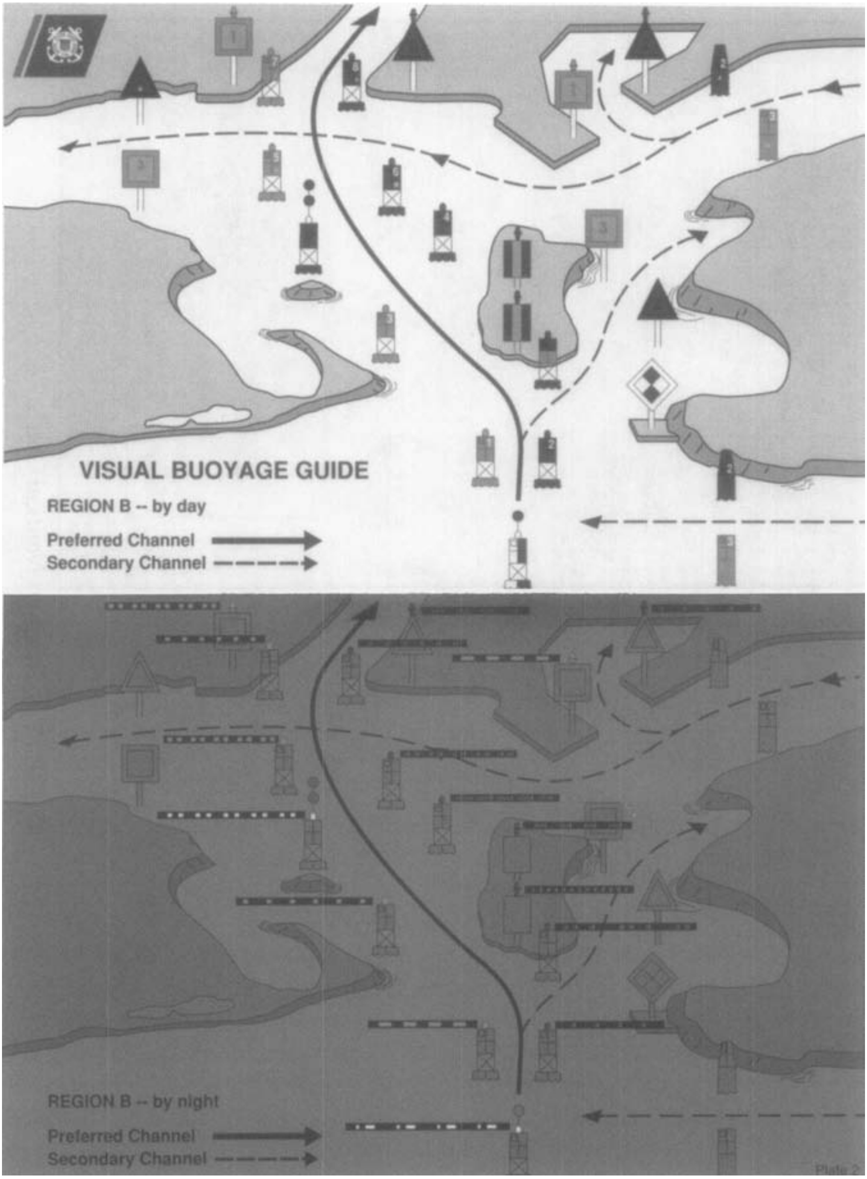


FIGURE 12-10. Visual Buoyage Guide.

the vessel bridge-to-bridge radio telephone frequency. Two exceptions to this are the use of Channel 16 on the Great Lakes and Channel 67 on the Lower Mississippi River (Baton Rouge area).

12.5.7 Vessel Routing Systems

Ship routing is a complex series of measures concerning routes aimed at reducing the risk of casualties. It includes traffic separation schemes, two-way routes, tracks, areas to be avoided, inshore traffic zones, and deepwater routes.

12.5.8 Vessel Traffic Services (VTS)

The Coast Guard VTS was authorized by Congress to prevent damage to, or the destruction or loss of any vessel, bridge, or other structure on or in the navigable waters of the United States. Using installed communications and surveillance equipment, the VTS is able to help prevent vessel collisions and in some cases, groundings, before they happen. In areas where a VTS has been established, restrictions and control of vessel movements during emergency situations should be coordinated through the Vessel Traffic Center. The Vessel Traffic Center can issue directions to control and supervise traffic during conditions of vessel congestion, adverse weather, reduced visibility, or other hazardous circumstances. The Vessel Traffic Center should have the authority to direct a vessel in an emergency to slow, stop, anchor, or otherwise proceed to avoid a dangerous situation; however, the master will at all times remain responsible for the safe and prudent maneuvering of the vessel. Locations of Vessel Traffic Management Services as of 1993 are:

a. Puget Sound, WA,
b. Straits of Juan De Fuca, WA,
c. Prince William Sound, AK,
d. Valdez Narrows, AK,
e. New Orleans, LA,
f. New York, NY,
g. Barwick Bay, LA, and
h. St. Marys River, MI.

12.6 RECREATION

12.6.1 Recreational Opportunities

Waterways have always been a recreational attraction to the genreal public. The pools behind navigation dams are usually large enough to accommodate recreational boating, fishing, and other water sports without interfering with commercial vessel traffic in the navigation channel. Therefore recreation should be included in any water project design. Some examples of common recreational activities are shown in Figures 12-11 through 12-14.

12.7 FISH PASSAGE AT LOCKS AND DAMS

12.7.1 General

The passage of migratory fish around navigation dams is a major consideration for the salmon runs on the Columbia and Snake Rivers. There are many factors that affect salmon survival success such as ocean and river fishing, dry and wet years, ocean conditions, quality of spawning areas, predators in the river system and ocean, and passage around or through navigation dams. This section addresses only passage at navigation dams.

12.7.2 Downstream Migration

The passage at dams of the downstream migrating salmon (usually three to four inches long) have been over the spillway or through the turbines. Spillway passage incurs the risk of nitrogen supersaturation disease which

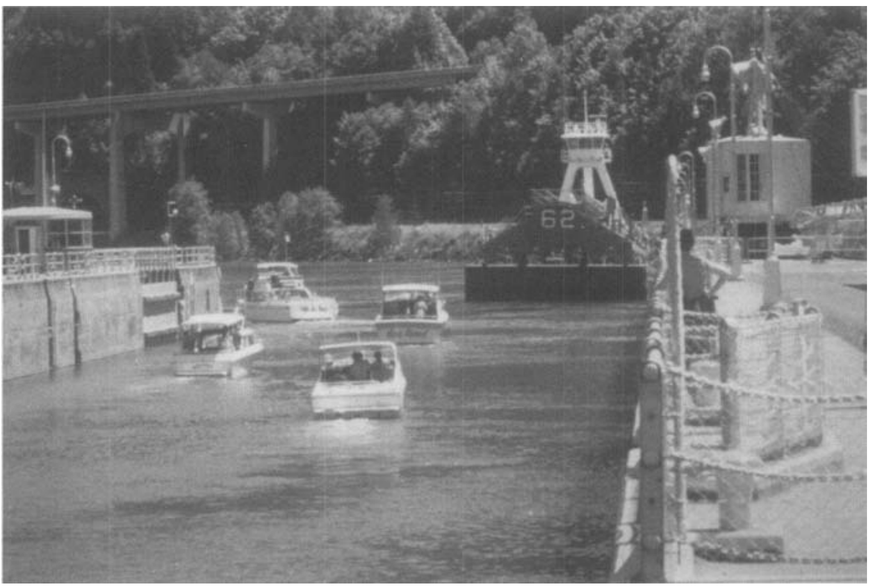


FIGURE 12-11. Recreational Boats Leaving Old Bonneville Lock, Columbia River.

FIGURE 12-12. Swimming and Picnicking Along the Columbia River.

FIGURE 12-13. Sturgeon Fishing Below John Day Lock and Dam, Columbia River.

FIGURE 12-14. *Wind Surfing in Bonneville Pool Near Hood River, OR.*

causes the formation of air bubbles in the young salmon (similar to the bends for deep sea divers). This occurs during high spillway discharges and can cause significant mortality. Turbine passage is relatively safe if the turbines do not operate in the cavitation range. However, predatory fish (squaw fish) and birds (seagulls) eat a significant number of the migrants as they exit the tailrace. Fish screens in the turbine intake have been used to divert the migrants into collection channels that bypass the turbine. These channels then carry the migrants to a discharge downstream from the dam or place them in a barge for transit past the other dams downstream. Figure 12-15 shows the element of the fish screen bypass system with a downstream discharge. Figure 12-16 shows a bypass system transportation channel to a barge-loading facility at Little Goose Dam on the Snake River. Over 20 million migrating salmon have been collected and transported by barges annually on the Columbia and Snake Rivers. Figure 12-17 shows a fish transportation barge. Additional information on fish passage at power houses is contained in a recent ASCE publication entitled *Guidelines for Design of Intakes for Hydroelectric Plants.*

12.7.3 Upstream Migration

Adult salmon returning from the ocean to spawn are faced with passage over navigation dams. The use of fish ladders has proven successful for this

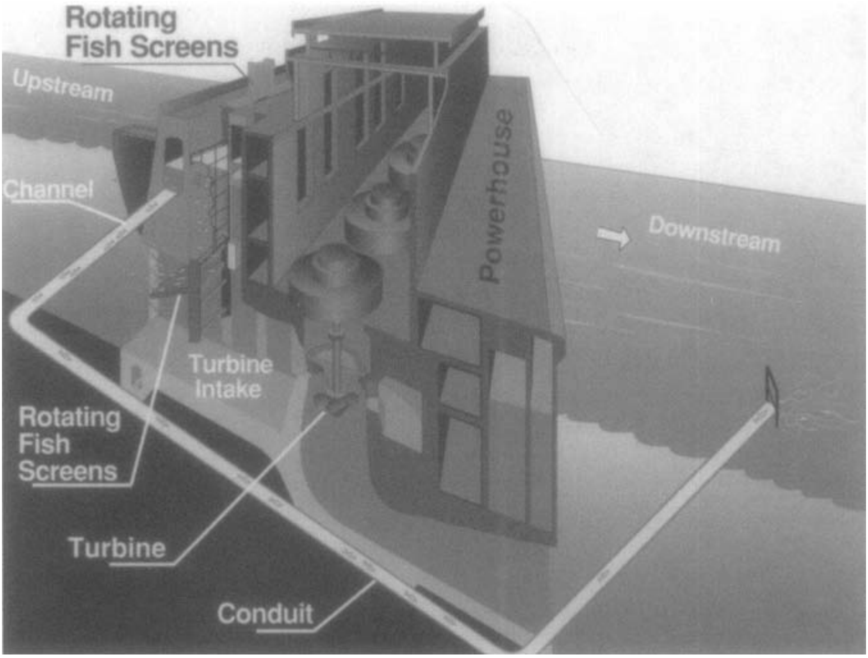


FIGURE 12-15. Fish Screen Bypass System.

passage. The ladder consists of a series of weirs and orifices in a concrete channel. About a one foot change in water level is provided between the pools formed by the weirs. The downstream entrances to the ladders are usually at the sides of powerhouses. The discharge from the ladder must be sufficient to attract the migrating salmon. Figure 12-18 shows the components of a fish ladder system. Figure 12-19 shows a fish ladder at Little Goose Dam. Figure 12-20 shows visitors observing a fish ladder passage near the fish-counting windows.

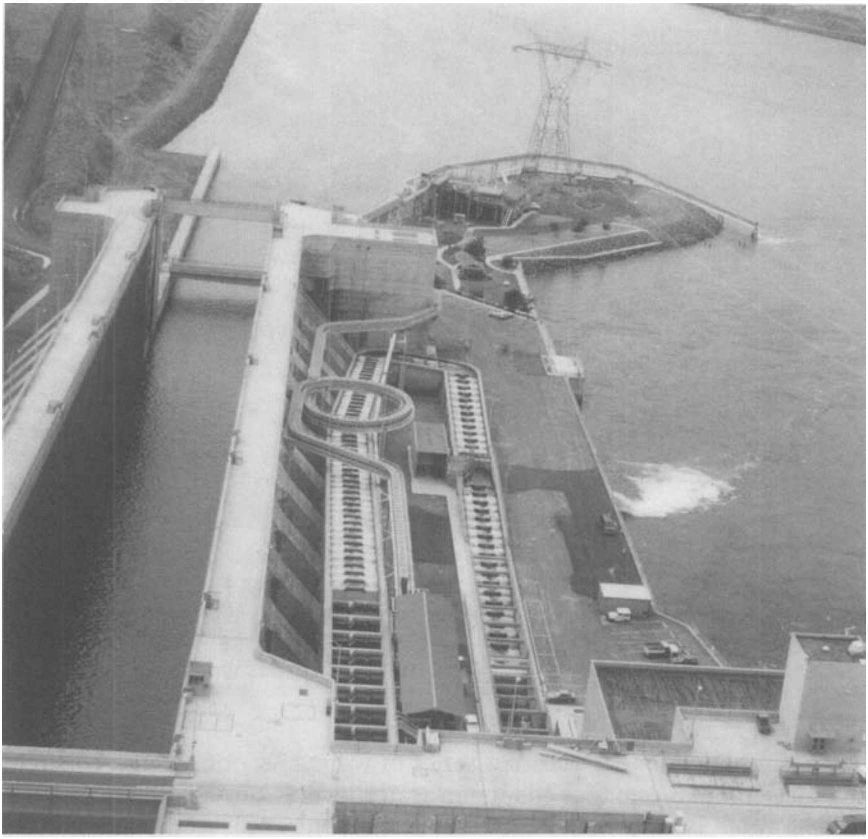


FIGURE 12-16. Fish Screen Bypass System at Little Goose Dam.

FIGURE 12-17. Fish Transportation Barge

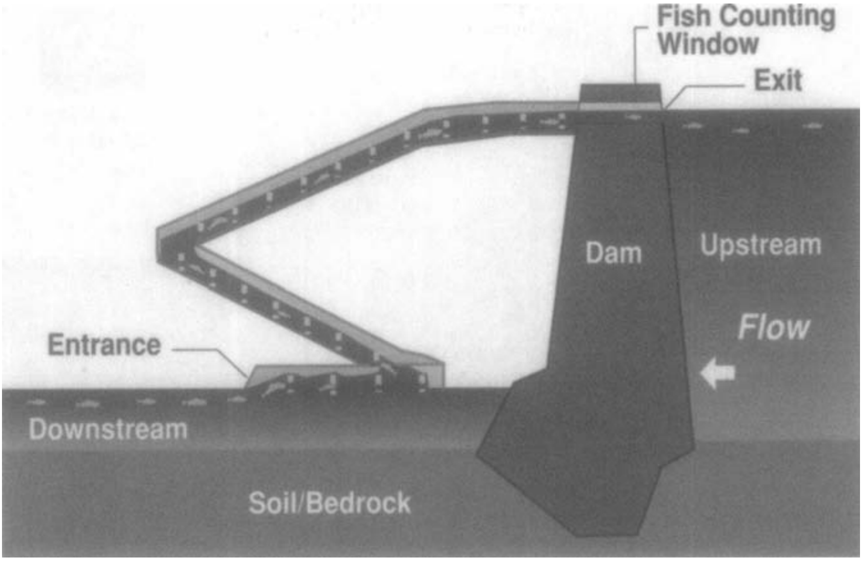


FIGURE 12-18. Components of a Fish Ladder System.

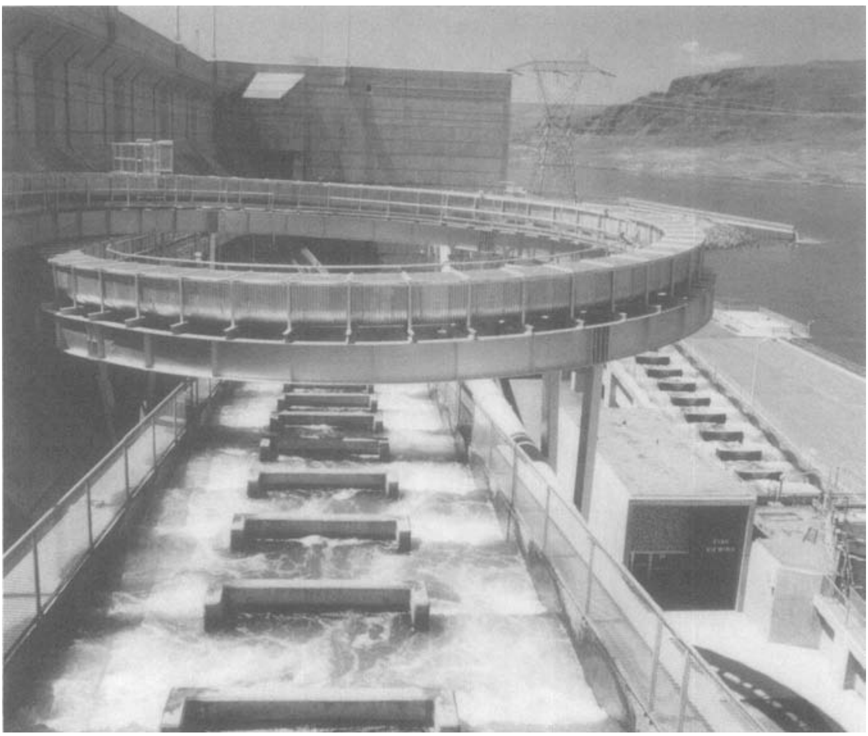


FIGURE 12-19. Fish Ladder at Little Goose Dam.

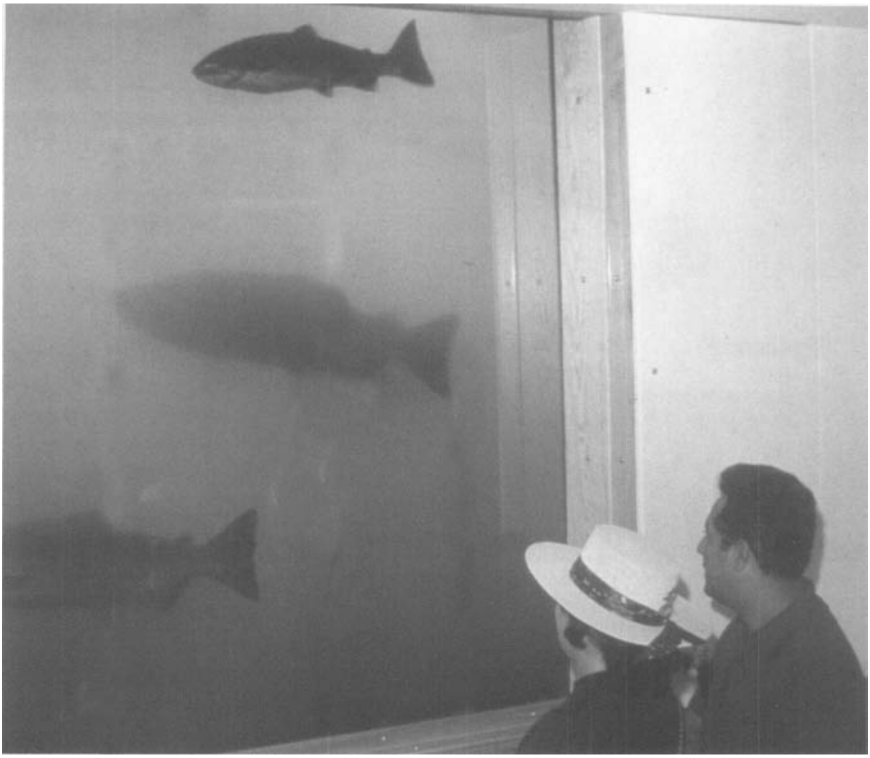


FIGURE 12-20. Visitor Observation of Fish Ladder.

Chapter 13

WINTER NAVIGATION

13.1 ICE PROBLEMS

13.1.1 Effects on Navigation

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

13.1.2 Effects on Structures

In addition to its effects on navigation, ice can cause damage to training and stabilization structures as well as mooring and docking facilities along the banks of the stream. Ice accumulation in lock approaches tends to block the entrance to the locks and could affect the operation of the lock gates. When a guard wall with ports is provided, ice and drift will tend to move into the lock approach and be trapped between the guard wall and adjacent bank and might have to be moved out or passed through the lock before traffic can be accommodated. Ice accumulation against partially closed spillway gates could render the gates inoperable and could result in flooding or overtopping of the dam and lock walls. The same effect could occur when ice jams or gorges develop in reaches between locks and dams or in open-river channels. Figure 13-1 shows the impact an ice jam can have on a navigation dam. The ice flow tore barges lose from their moorings and carried them into a dam. The tug in the foreground was trying to clear the ice but experienced problems and was partially submerged. A cable from the tug to the lock wall was an attempt to save the vessel.

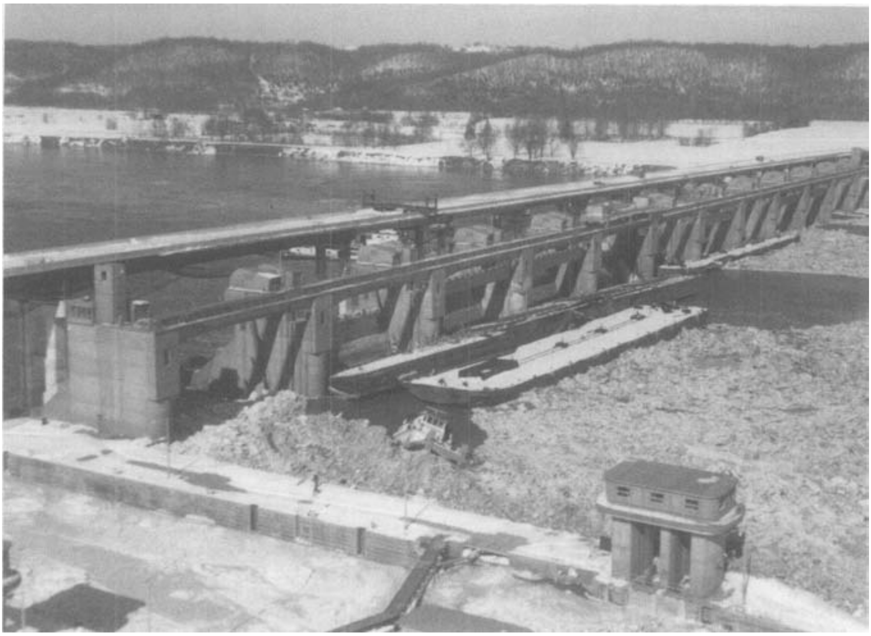


FIGURE 13-1. Ice Flow at a Navigation Dam.

13.1.3 Design Considerations

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

1. air bubbler screen or ice boom designed to divert ice and debris away from the lock approach;
2. high air flow screens in gate recesses;
3. lock emergency gates designed and maintained for passing ice and debris;
4. means of preventing ice formation in the area of the lock miter gates, the lock filling-and-emptying valves, lock walls, and emergency bulkhead latching devices;
5. protection from excessive scour downstream of the spillway during uneven operation of the gates to pass ice;
6. heating cables or pipes in lock walls;
7. chemical coating of lock walls to reduce ice adhesion;
8. elimination of sharp bends in the channel and constricted reaches where ice jams or gorges might develop; and

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

13.1.4 Ice Control Methods

It is desirable and often essential to continue operation of navigation dams and spillways during winter. Traffic may be curtailed or even stopped on the waterway but provision must be made to pass winter flows and to handle ice during winter and at breakup. Designers must consider ice passage procedures, possible ice retention, ice forces on the structures, and icing problems leading to blocking of moving parts or simply excess weight (Figure 13-1). Provisions to move ice past or through dams have been many and varied and none have met with perfect success. At some locations, it is preferable to retain the ice in the upstream pool, whereas at others an ice-passing capability is necessary. Spillway gates should be as wide as practicable to minimize arching across the openings. The primary factor controlling ice passage appears to be the velocity of the approaching ice. When the velocity is great enough, the flows are broken and pass through spillway bays. Passage of ice through a submerged outlet requires sufficient velocity to entrain the ice into the flow. Therefore, to maintain pool during periods of low flow, it is preferable to pass ice over the top of gates in a skimming type mode. At low flows ice can be passed with one or more gates open at a time and arching broken by alternating gate openings.

13.2 ICE JAMS

13.2.1 Introduction

When the ice goes out of the rivers it often jams and causes flooding of fields and homes. The ice floating upstream from a jam can destroy houses, bury roads, and collect on fields, delaying spring plowing and planting.

13.2.2 Discussion

There are two processes that either alone or in concert are responsible for breakup. Ice strength gradually deteriorates in the Spring when higher sun angles and higher air temperatures melt snow from the ice surface, forming a water layer. This water layer absorbs more solar radiation, causing subsequent melt along the crystal boundaries. If not disturbed by other factors, the ice will melt in place.

In rivers, the current flow beneath the ice is a second factor. In fact, water flow is the sole cause of the midwinter breakups that can lead to the most destructive ice jams. Any increase in water flow down the river will raise the ice level and break it loose from the shore. If the river discharge stays high because of rain or snow melt on the upper sections of the watershed, the higher flow will move the ice downstream. As it moves, the ice breaks up; the size of the pieces depends on the distance they move and the degree to which the ice strength has deteriorated. As might be expected, the ice in those reaches with steeper slopes and higher current velocities will go out first. When the moving ice hits the fixed ice in a slow, flat reach, it may break up the stationary ice and carry it along, or form a jam. Ice jams occur in two basic forms, the dry jam and the simple jam. They are essentially identical except that in a dry jam the ice is grounded, restricting water flow to a greater degree than a simple jam.

Predicting the time or even the probability of an ice jam occurring is still uncertain. However, there are a number of typical locations where a jam will form. As mentioned earlier, any section of a river where the slope decreases is a possible location. During freezeup the slower moving reaches freeze first, and so will have a thicker ice cover come breakup. Another possible location might be a constriction in the channel, either natural, such as at a bend or at islands, or at manmade features such as bridge abutments and midstream piers. A third typical location is a shallow reach where the ice can freeze to bottom bars or boulders and will not be lifted and moved by the increased water flow.

Once the ice is stopped at any location the jam thickens rapidly, primarily by ice blocks turning under existing surface ice. The net result is a very rapid constriction of the channel and subsequent backing up of the upstream flow.

Flooding from an ice jam occurs very quickly. The situation is not like a normal open water flood when the channel is not large enough for the flow. Instead the channel is completely blocked. Normal backwater calculations based upon stage level recorders are meaningless. Suddenly there is a new dam in the river, albeit a leaky and temporary one, which is creating a lake and has no convenient spillway. The best time to try to move a jam is while the water pressure behind it is still high and the flow rates are adequate to carry the ice downstream. If the jam occurs in midwinter and a cold spell reduces the flow before the jam moves on, it can settle on the bottom and remain for the rest of the winter, creating a potential hazard. During the rest of the winter a new ice cover can form upstream, and when the Spring breakup comes the new ice cover will be stopped at the old jam; flooding is almost a certainty. The need to free or remove some ice jams is thus obvious, both in cases when flooding is actually present and in cases when the potential for subsequent flooding exists.

13.2.3 Methods of Ice Jam Removal

There are four different methods for removing ice jams or alleviating ice jam problems, and each has its advantages and drawbacks. These are mechanical removal, dusting, blasting, and the use of icebreaking ships. It is important to remember that ice loosened in a stream may jam elsewhere. A decision must be made. Is it best to move the jam and take the possible financial responsibility for downstream damage, or to accept the potential damages caused by the jam as is? Once the decision has been made to try to remove the jam, which approach will be the most effective?

13.2.3.1 Mechanical Removal. Removing the jam mechanically, for lack of a better term, means simply taking the ice out of the stream bed and placing it elsewhere. This, of course, eliminates any downstream problems but it is neither cheap nor fast. In February, 1978 it cost approximately $11,500 to make a 2,600 ft long channel with one backhoe. The approach is further limited to dry jams in relatively shallow streams. In other words, this approach is used generally for midwinter jams on small streams after the flooding has receded. The idea is to create a small channel within the jam by using mechanical equipment such as bulldozers, backhoes, or draglines. When the ice blocks are small and thin, mechanical clearing does not present too great a problem. When the blocks are around 10 × 10 × 2 ft or larger, small equipment is generally inadequate. Each site is different, so that equipment and methods used are up to the operator, who must be aware of the problems of power lines, poor bottom, and access. An immediate problem is disposing of the ice. Usually it can be pushed to each side, leaving a channel about one third the normal river width. In reaches where the channel has been severely restricted by manmade works, it may be necessary to remove all the ice.

13.2.3.2 Dusting. A second method for alleviating ice jam problems is the use of dust. By dust we mean any dark substance that can be spread on the ice in a thin layer to absorb solar radiation and thereby hasten the deterioration process. This method is used primarily to alleviate possible jam conditions before the fact. The rough surface of an actual jam creates so many shadows that the dust is not effective. Ideally, the dust should be applied as early as possible but after the last snowfall. In general, any reach with an ice cover that regularly stops the ice run and causes a jam could be weakened in this manner.

Dusting involves spreading (as evenly as possible) a dust leaf mulch or sand layer and letting the sun provide the energy. Thus time is involved as well as the higher sun angles in the late Spring and good luck in avoiding snow storms that would cover the dust. Agricultural aircraft generally apply the dust, which keeps costs fairly low. For example it cost 34.9 cents in 1970 per lineal foot (100 ft wide) to dust a remote section of Alaska. The particle size can vary, depending on what is available.

A logical offshoot of dusting is to pump water and bottom materials onto the ice surface. This is limited to streams with silt or sand bottoms and is 10 times more expensive than aerial dusting. However, the approach does have application where the stream is too narrow or sinuous for aerial work, or where environmental considerations preclude adding material to the stream.

13.2.3.3 ***Blasting.*** The third method is blasting the jam. For immediate flood relief this is probably the most effective. The primary purpose of the blasting is to loosen the ice. However, enough flow must be coming through the jam to float the ice downstream. Thus a prerequisite to blasting is an ice-free reach downstream where the ice can go, either all the way down the river, or to a spot where it will not jam (or, if it does, where the jam will not cause any appreciable damage). Unfortunately, jams have been blasted without regard to downstream problems. Successful blasting takes time and careful planning.

The ideal time to blast a jam is just after it has formed. In actuality, a jam is never blasted this quickly because a blasting crew and governmental approval cannot be mobilized until the jam is well formed and flooding has begun. If the flow has dropped because of cold weather or has moved into another channel so that after a blast there will not be enough water to carry the loosened ice downstream, the blasting should be canceled.

If the decision has been made to blast, there are a number of procedures learned from experience that can lead to a cheaper and more successful job. Each charge, if placed under the ice, will blow a crater or circle in the ice with a diameter that is related to the weight of the explosive. Figure 13-2 gives this relationship. A handy charge size for most jobs is around 40 lbs, which gives a diameter of close to 40 ft. Experience has shown that 2 more or less parallel rows of charges, set close enough so the craters intersect, give the best result. If it is possible to locate the thalweg or deepest part of the river, the blasting line should be along it. This creates an open channel with good flow depths that is wide enough to preclude most secondary jamming. The charges must be placed in the water below the ice cover. This is extremely important since the driving force is apparently the large gas bubble resulting from the blast, and not the shock waves. The charges must be weighted to sink but also roped to the surface to keep them from being carried downstream by the current.

Blasting is not a quick, easy solution. It requires some planning to locate and acquire the explosive, the equipment to make holes to place the charges, and manpower. At all times when the crew is working on the jam, a lookout should be on duty some thousand feet upstream to sound the alarm if the jam lets go by itself. At least 2 men are required to drill holes and, depending on the roughness of the surface, at least 4 more to carry the charges to the holes. Add a blaster, a supervisor, and 2 men to load the charges and you have a crew of 11 people. With good luck this crew can blast 2 rows of charges along about a half mile of river per day, possibly more when a routine has been established.

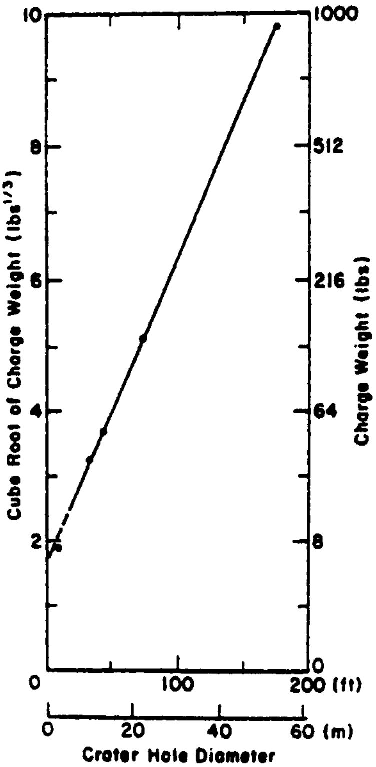


FIGURE 13-2. Relationship of Explosive Weight to Crater Hole (Good for Ice 1 to 10 ft Thick).

A formal safety plan covering all operations is necessary. It should comply with both local and Federal regulations. Such matters as person in charge, communication, transportation, warning personnel, and so on should be fully covered.

13.2.3.4 Icebreakers. The fourth method of removing jams is only usable in a few rivers. When the channel depth is sufficient and the ships available, icebreakers are certainly the easiest, safest, and cheapest way to break up a jam. This operation is carried out by the captains, who are responsible for the safety of their ships, so little more needs to be said regarding safe operations. If two ships are available, they work best in echelon (staggered one behind and to the side of the other), starting from the downstream end of the jam. The following ship has to be careful to ensure an equal width channel. If it crosses the path of the leader, the resulting narrow section will inevitably cause a jam and the downstream channel will no longer keep itself clear. Occasionally, if circumstances permit, an icebreaker can work in conjunction with blasting. The propeller wash and wave action of the ship will clear the ice loosened by the blasting faster, and the ship will offer a factor of safety for the people on the ice. A combined operation like this will require extra cooperation as well as good communication. When the jam is very thick, two towboats of essentially equal power have been used together. They mate-up bow to bow, and while the propeller wash of one boat loosens and erodes the ice, the second boat holds the first in position. This operation takes a great deal of skill and coordination between the pilots.

13.3 ICE BREAKING

13.3.1 Introduction

A vessel navigating in ice must tolerate stresses imposed by an environment that is not encountered by regular shipping. The vessel's form, power, structure, and propulsion system must be designed to withstand these stresses. In addition, the effect of the vessel on the environment must be considered as well as the effect of the environment on the vessel.

13.3.2 Environment

In Winter a vessel may encounter plate ice, brash ice, frazil ice, pressurized ice, a pressure ridge, ice with snow cover, or a combination of all these forms. The easiest of these to deal with is plate ice—homogeneous ice with fairly uniform thickness. A properly designed vessel can travel through plate ice, up to some limiting thickness, with very little effort.

The second type of ice, brash, is broken ice that fills a shipping channel with pieces up to 6 ft in diameter. Brash ice may fill the channel completely or partially and it can be unconsolidated or consolidated and refrozen. This type of ice, because of its lack of homogeneity, restricts vessel movement differently than does plate ice.

The third type of ice is frazil ice. Frazil is highly cohesive in its active state. If the water velocity slows beyond a certain point, the frazil crystals can agglomerate and form a mush that can eventually solidify and block the total depth of the channel.

All these forms of ice can restrict traffic further if the ice sheet is under lateral pressure. Lateral pressure can be caused by wind or water currents. Ice sheets can also push over each other and form a pressure ridge. Such a ridge can grow to extreme depths and virtually block a channel.

The various types of ice described can also be found with a snow cover. A snow cover does not affect the mechanical properties of the ice to any great extent; however, a snow layer increases the friction between the ice and the ship's hull.

13.3.3 Vessel Shape

Most vessels are designed to maximize the volume of cargo that they can carry; they tend to be rectangular with minimum curvature of the hull, the extreme example being the rectangular barge. Icebreakers are specifically designed for breaking and clearing ice, and have angled bows, special shapes, and are usually highly powered. Between these extremes are the blunt-bowed ore carriers, the raked-bowed barges, and passenger vessels.

The resistance a vessel encounters in ice depends on its hull shape. The efficiency of a particular hull depends on the forces involved in breaking and clearing ice. Basically, a vessel breaks the ice by riding on top of it, causing the ice sheet to fail from tension in the lower and upper layers. After the ship breaks the ice sheet, it must clear the ice fragments from the channel. This is done by pushing the fragments down or to the side. The resistance of the ice to breaking and clearing is a function of the friction between the vessel and the ice and of the lateral pressure in the ice.

The resistance encountered by the vessel increases as the width and length of the vessel increase, as the thickness and strength of the ice increase, as the velocity of the vessel increases, as the friction between the ship and the ice increases, and as the lateral pressure in the ice increases.

In the case of a tug and towed or tug-pushed barges, the shape of the forward part of the hull has the largest effect on the resistance. A wide vessel with a blunt bow that has a very rough surface will encounter extreme resistance. If the bow is so blunt that the ice cannot pass to the side or below the vessel, the ice will pile up in front of the vessel, forming its own "bow" shape, and will eventually cause such high resistance that the vessel may be unable to move.

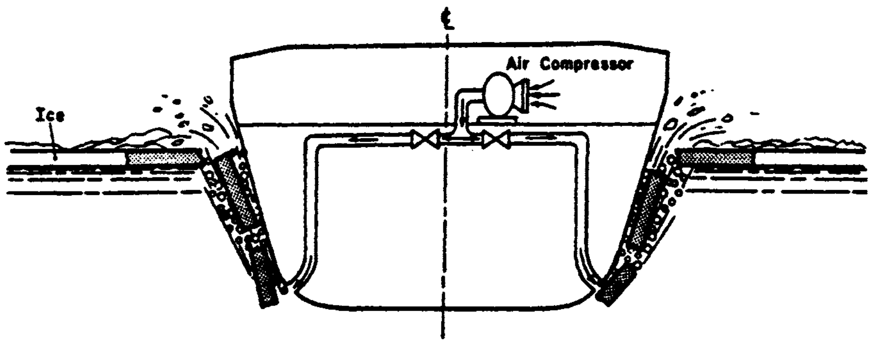
13.3.4 Auxiliary Ice Breaking Devices

Several different methods are being developed to facilitate ice breaking and ice navigation. The most promising methods are low friction hull coating, hull bubbler systems, air cushion vehicles, and ice cutting vehicles.

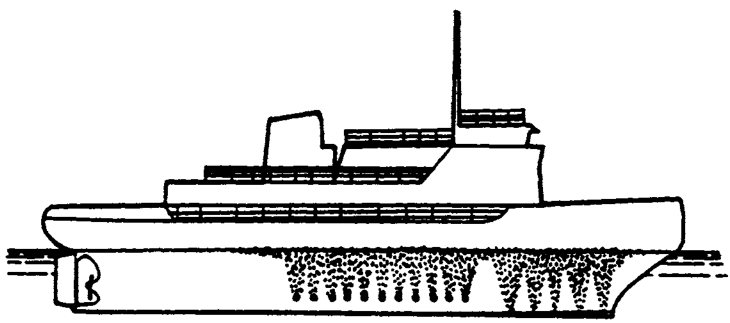
These systems are in various stages of evaluation and development by the US Army Corps of Engineers, the US Coast Guard, the Canadian Coast Guard, and ice researchers, as well as Finland and the USSR.

Polyurethane and epoxy (nonsolvented) coatings proved to be the most effective in friction reduction and coating endurance for the hull of an ice breaking vessel.

Hull bubbler systems have been installed on several European icebreakers and on the latest USCG small lake icebreakers. Bubbler systems work by interposing air and water between the ice and the hull of the vessel. Figure 13-3(a) is a schematic of the bubbler system. Figure 13-3(b) depicts its deployment on a new USCG icebreaker.



a. Schematic of bubbler system.



b. Bubbler installed on icebreaker.

FIGURE 13-3. Hull Air Bubbler System: (a) Schematic of Bubbler System; (b) Bubbler Installed on Icebreaker.

The air cushion vehicle (ACV) is the most dramatic contribution of modern technology to ice breaking. The vehicles can skim over the ice and break it at speeds of 3 to 20 miles per hour (mph). The ice breaking occurs both at low speeds of advance as shown in Figure 13-4 (top) and at higher speeds of advance (bottom). At high speeds, the critical speed of the craft deflects the ice sheet to the ice breaking point. At low speeds, the air cushion extends under the ice, displacing the supporting water. Deprived of its support, the ice sheet fails under the pressure of the air cushion. Tests conducted by the Canadian Coast Guard indicate that an ACV can break ice whose thickness is 90% of the cushion pressure expressed in inches of water. The ACV has significant potential for aiding ice-jam flood control in shallow rivers and estuaries where vessel draft is limited. An ACV placed in front of a conventional icebreaker will increase its effectiveness.

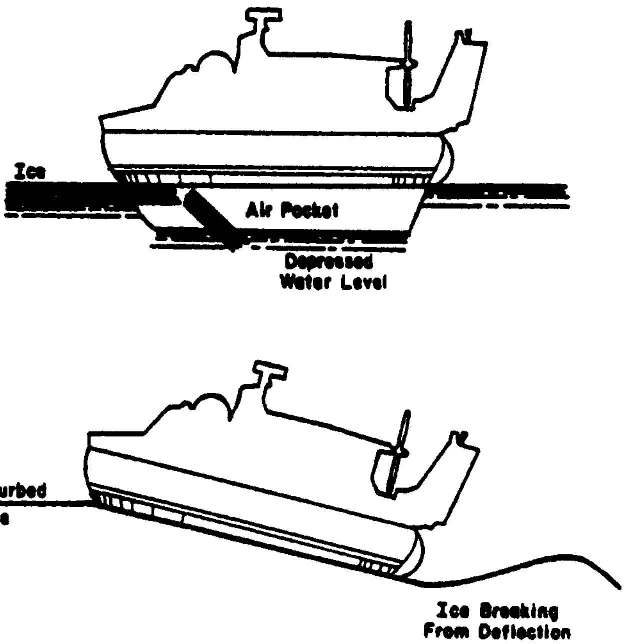


FIGURE 13-4. *Air Cushion Vehicle. Top: Low Speed Ice Breaking. Bottom: High Speed Ice Breaking.*

Another device, which is used in the Soviet Union and is being evaluated in the US, is the ice cutting vehicle. Such a vehicle cuts the ice with some apparatus such as a circular saw or a high-pressure water jet. The weakened ice is then either conveyed up onto the vehicle and thrown over the side, or deflected beneath and to the side of the vehicle by underwater ice guides. Figures 13-5 and 13-6 depict conceptual sketches of two possible ice cutting and clearing devices. These devices could be used to keep channels in narrow rivers between locks and dams clear of ice.

13.3.5 Summary

Vessels operating in ice must be given special consideration if they are to operate safely and efficiently. The vessel must have the power and structure to overcome the resistance and loads imposed by the ice environment.

Properly shaped vessels with adequate power can break the ice that is encountered on lakes and rivers. The primary problem is not so much sheet ice, but brash and frazil ice that can fill the channel and cause unusually high resistance due to the friction between the ice and hull surface. This problem can be mitigated somewhat by a low-friction, high-wear coating.

To enhance winter navigation on lakes and rivers, extensive assistance is required in the form of ice breaking, ice clearing, ice control, and towing or

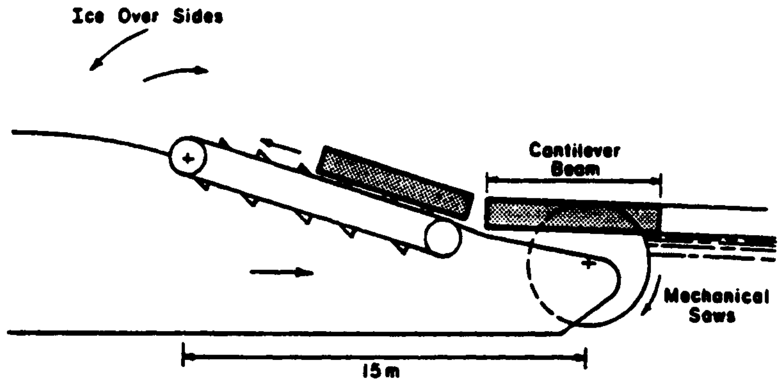


FIGURE 13-5. *Ice Cutter, Type 1.*

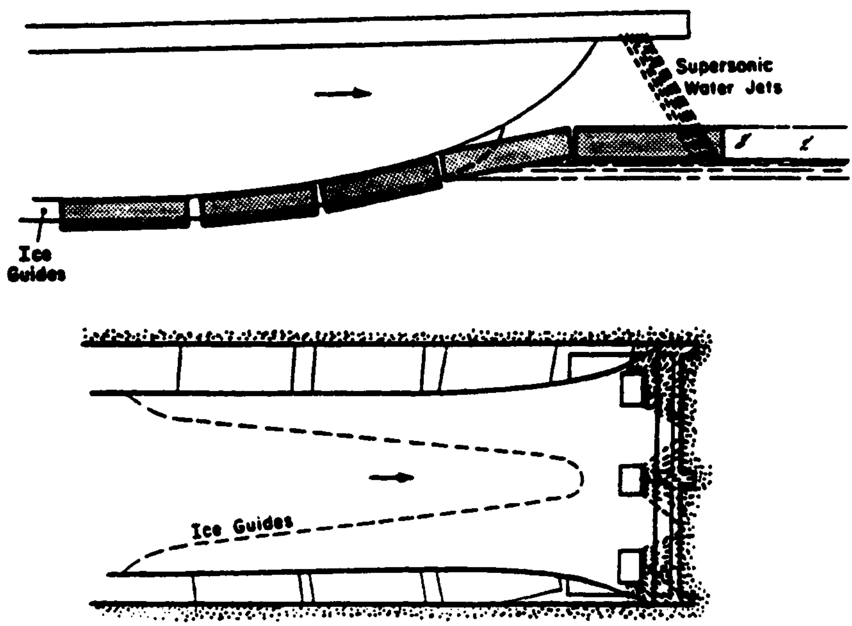


FIGURE 13-6. *Ice Cutter, Type 2.*

kedging. This assistance usually is provided by a combination of government agencies (US Army Corps of Engineers and US Coast Guard) and private industry.

13.4 ICE ADHESION

13.4.1 Introduction

The extension of the navigation season into the Winter months on the Great Lakes creates ice problems at the navigation locks. Ice can adhere to lock gates, interfering with their opening and closing; to trash racks and water inlets, clogging them; and on lock walls, building up an ice collar at and below the high pool level that can interfere with ship passage. Ice collars occur at the 110 ft wide Poe Lock at Sault Ste. Marie, MI. Ships of the *Presque Isle* and *Roger Blough* class with their 105 ft beams encounter problems when the ice buildup along the walls becomes greater than 2.5 ft on each wall. On the rivers the standardization of barge width and their square bows minimize this problem. Prior to the development of the ice cutting saw and the copolymer coating, a number of methods were used to overcome this problem, with varying degrees of success. Steam hoses work well but are extremely slow and require many man-hours. Backhoes have been used to scrape off the ice collar. This is faster than using steam, but still slow. Since the operator cannot see what he or she is doing, some ice may be missed or scraped too deep, damaging the lock wall. A high pressure water jet would be able to cut off the ice, but the jet is noisy and somewhat dangerous and the pressure pump both expensive and difficult to maintain. The present solution, using an air screen, copolymer coating, and ice cutting saw, does the job efficiently with few operational and maintenance problems.

13.4.2 Ice Cutting Saw

The Corps of Engineers Cold Regions Research and Engineering Laboratory designed and assembled a mechanical cutting system to remove the ice collars. The unit consists of 2 parts: the cutting system and the drive and propulsion system. The drive and propulsion system is a 65 horsepower, 4-wheel-drive tractor, originally manufactured as a trencher (the tractor can be purchased without the trencher attachment). The drive line for the trencher was modified to accommodate the cutting system by extending the drive shaft and attaching a drive sprocket to its end. While in the cutting mode, the engine powers the shaft and sprocket directly, and the drive wheels indirectly through a separate hydraulic drive system so cutting power and propulsion power can be independently controlled.

The cutting system is one used in the coal industry, a thin, 3.5 in. kerf cutter manufactured at the Bowdil Company of Canton, OH. It consists of a rugged bar and chain with cutting bits attached. The bar is 9.5 in. wide to the chain guide, 1.5 in. thick and 15.9 ft long, and is attached to the drive shaft housing. Movement of the bar is hydraulically controlled. Different kerf and bar thicknesses have been used, but earlier tests showed that a narrow logging saw was too flexible.

The bar is grooved to accommodate the sprocket drive chain and cutting bits and has a roller nose tip to reduce friction and wear. Chain tension is controlled by a high-pressure hydraulic cylinder capable of exerting 1,800 lbs at 10,000 psi. The bar and chain hang about 30 in. past the side of the tractor and the drive wheels (see Figure 13-7).

13.4.3 Operation of the Ice Cutting Saw

When a problem ice collar has built up, the esplanade along the lock wall is cleared of snow. The tractor is then positioned with the right wheels close

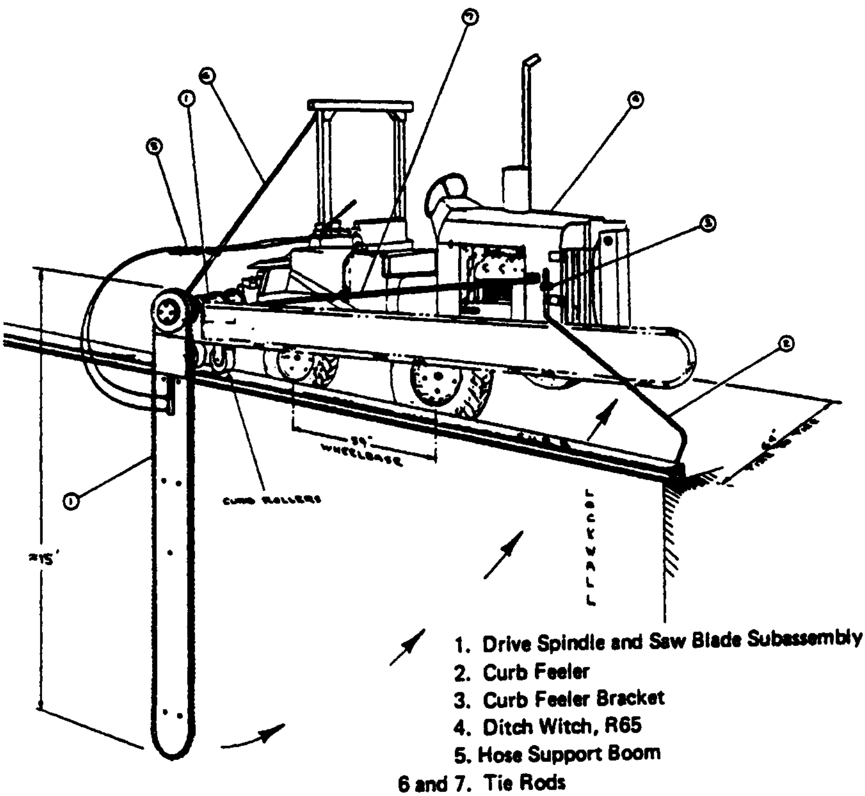


FIGURE 13-7. Schematic of Ice Cutting Saw.

to the curbing along the wall so that there is about 1.5 in. of clearance between the wall and the bar and chain. A spacer on the wall side of the bar prevents the cutters from damaging the wall. A guide marker located off the right front wheel is positioned and set so the driver can maintain the proper position by keeping the marker and the reference point (top of curb) aligned. Looked at from the driver's point of view, the chain rotates clockwise with the tension cutting side on top of the bar. To start a slot for the bar, the underside of the saw is used until the tip cuts completely through the collar. The slot is cut with the tractor stationary. Once a slot is cut through, the bar is placed in a forward position about 70 degrees from the horizontal. Full throttle operation in third gear produces a chain speed of 380 ft per min., although chain speeds of up to 510 ft per min. are possible in fourth gear. A traverse speed of over 10 ft per min. can be maintained while cutting ice collars 6 to 8 ft deep by operating the transmission in third gear at full throttle.

13.4.4 Copolymer Coating

A chemical coating that reduces the adhesive force between the coated surface and the ice can also help solve icing problems, although the ideal material would prevent ice formation altogether. The coating that was developed does not prevent ice formation, but makes removal of ice from coated surfaces much easier. The basic material is a long chain copolymer compound made up of polycarbonates and polysiloxanes. The copolymer coating should not be applied to a concrete surface unless it is certain that the concrete behind the coating can resist frost action in a critically saturated condition.

13.4.5 Application of the Copolymer Coating

Extensive testing has shown a solution of the copolymer LR 5630, silicone oil, and toluene to be the most effective coating. The mixture is highly volatile and leaves a thin coat of the copolymer and silicone on the surface to which it is applied. The surface to be coated must be clean and dry. For concrete and metal surfaces (bare and painted), steam cleaning is sufficient. In a case where the lock walls were heavily coated with oil and algae, a detergent was added to the water of the steam cleaner. Once the surface is clean and dry, the copolymer solution can be applied with an airless spray gun system. A single pass will deposit a coat 1 to 2 mils thick. Three coats are recommended for a coating thickness of about 5 mils.

Some care will have to be taken when mixing the solution. Toluene is a combustible material so the mixer should not have an electric motor. An air-operated drill motor turning a rod with mixer blades has worked satisfactorily. The fumes may also be a health hazard so a well-ventilated mixing area should be used. A 55 gallon drum fitted with a bracket to hold the drill motor is a suitable mixing container where batches up to 40 gallons can easily be handled. The toluene and the silicone oil should be placed in the container first. Then the mixer is started and the copolymer powder slowly added. The solution should be mixed until all solids are dissolved and then transferred to a storage container.

The value of an undercoating to copolymer adhesion on concrete surfaces that are worn and rough is being tested. An epoxy-type coating has been used as a filler over the rough concrete. We hope the undercoating will provide a surface to which the copolymer can readily adhere. Trials of the undercoating and copolymer are being evaluated at the Poe Lock, at the St. Marys Falls Canal, at Sault Ste. Marie, MI, at Lock No. 4 on the Allegheny River, and at the Starved Rock Lock on the Illinois River. Maintenance and frequency of recoating requirements are being monitored. The original coating on the Poe Lock is now three years old and still in good condition.

13.4.6 Options for New Locks or Lock Rehabilitation

During new construction or rehabilitation much of the compressed air piping now used for air screens and gate recess clearing could be cast in the concrete, getting it out of the way and protected. Piping for wall heating using water or glycol could be cast in place. At a lock, piping should go from high pool level down perhaps 6 ft. The rest of the wall is kept warm by the water if the lock is kept close to high pool level when not in use.

13.5 ICE CONTROL

13.5.1 Introduction

Ice control is the practice of holding ice in place or directing its growth and movement. Booms, small islands, bubblers, and other structures have been used for this purpose. The ice boom is a barrier made from floating pontoons or timbers held in place by chain and wire rope. Booms are also used to initiate an ice cover, thereby minimizing frazil ice generation. Bubblers are used to bring up warmer water from some depth to melt ice or at least retard its growth. An offshoot of the bubblers is the air screen, which by releasing large quantities of air at some depth induces a diverging current at the surface. This surface current prevents ice and debris from passing; however, its use is restricted to sites with low flow velocity. Ice piers have been constructed on the Ohio River above Cincinnati. These are simply large bridge piers or cells placed fairly close together to slow or stop ice flow. A tow may take shelter below these piers during an ice run.

13.5.2 Ice Booms

Ice booms have been installed primarily by hydroelectric power companies to minimize the volume of ice impinging on their trash racks, to minimize the formation of frazil ice, and to keep head losses to a minimum. The boom or series of booms collects floating ice and accelerates the formation of an ice cover upstream. When the discharge is controlled, as at a power station, a decrease in flow will further accelerate ice cover formation. Booms have been installed to restrain and thereby minimize ice contribution to ice jams or ice pileups on shore that can block water intakes. Lastly, booms have been used as a navigation aid by holding ice broken by ship passage in place so that it does not flow downstream and block narrow channels. Booms are not intended to restrain ice at breakup.

13.5.3 Boom Configuration

As shown in Figure 13-8, booms have been built in many configurations; some crossing the entire width, some with a gap to permit navigation, some restraining ice on only one side of the channel, and some more or less in the middle of a lake with open passage around both ends. The common boom section in use today is made from a number of $1 \times 2 \times 20$ ft timbers. These are chained to and support a wire rope connected to bottom anchors and usually to the shore at 100 to 400 ft intervals, depending on the load.

13.5.4 Site Considerations

Locations where ice booms have been used successfully share common characteristics. The discharge is fairly constant and there are no abrupt changes in cross-sectional area. A boom can be used in fairly deep water, anywhere from 10 to 60 ft or more. Successful booms require a stable river bottom, that is, one unaltered by sediment transport. If water velocities are too high, the ice floes can be turned under the boom. A location that sustains a natural ice cover, even if only occasionally, is generally feasible. The Froude number, based on flow depth, should be less than the nominal value of 0.08. A higher value indicates that ice thickening conditions may be present. The surface velocity of the river should be less than 2.25 ft per second (fps) to achieve good boom performance. At some sites, modeling may be necessary to select the optimum location.

13.5.5 Design Considerations

There are a number of loads and other actions that must be considered when designing an ice boom. Fluctuations in water level may allow the boom to pound on the bottom if it is too close to shore. Because of the resulting damage, this should be avoided.

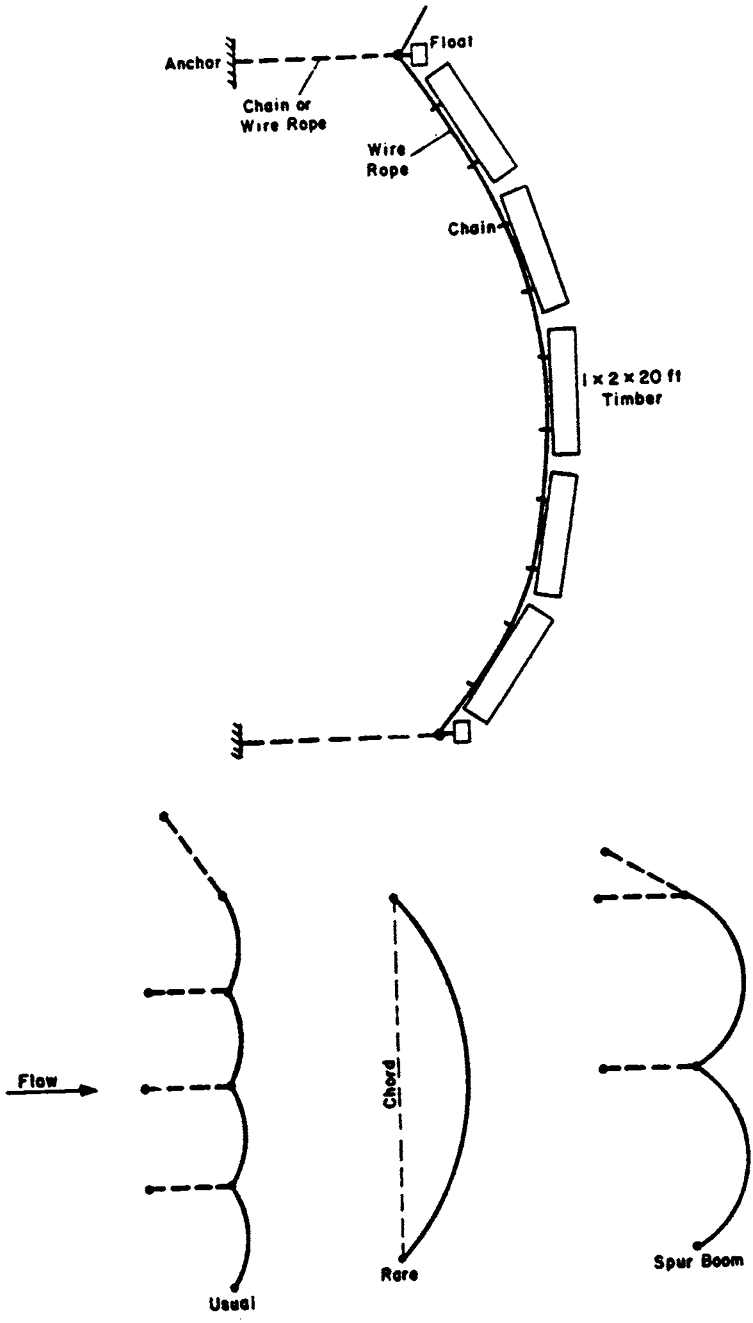


FIGURE 13-8. Ice Boom Configurations.

Generally, the amount of ice that will bleed through a small gap between the end of the boom and shore is negligible. As apparent in the example problem, bottom conditions must be adequate for a substantial anchoring force. The total force on the ice boom is

$$f_i = f_w \pm f_a + f_g + f_p + f_k - f_s \pm f_v$$

where

f_w = water drag force on ice cover
f_a = wind drag force on ice cover
f_g = gravity force
f_p = water flow pressure at beginning of ice cover
f_k = impact forces from collecting ice floes
f_s = shear force between ice and shore
f_v = forces resulting from vessel/ice/structure interaction.

Most of these are self-explanatory or are explained more fully in the example problem. The shear force between the ice and shore f_s is presently indeterminate, but field observations have shown that ice loads come from the area upstream of the boom which is four or five times the river width. In other words the drag forces on the ice cover in excess of four or five B (the river width) are taken by the shore and are not felt by the boom. Boom pontoon stability should also be considered.

The restraint characteristics of a generally applicable timber configuration are given in Figure 13-9. The timber has a no-load submergence of 0.75 and a connection point upstream on the bottom. The curves show that the ice restraint force will be about the same for anchors pulling horizontally and anchors pulling downward at a small angle $\alpha = \tan^{-1} 0.08$ until the tilt β of the timber is about 35 degrees. At 35 degrees the $\alpha = 0$ curve changes slope rapidly and diverges from the $\alpha = \tan^{-1} 0.08$ curve. The physical problem here is that as the tilt angle nears 40 to 45 degrees the ice can slide over the timber; this tends to make this portion of the curves unreliable. The restraint capacity of a boom timber increases by the cube of the timber width.

The override effect can be and is used as a protective measure that helps the boom survive the high loads imposed at breakup. Generally the timber booms have a load capacity of about 50 lbs per lineal foot or more, depending upon connections. There are double pontoons that have load capacity of over 500 lbs per lineal foot. The load capacity depends on the buoyancy, the righting couple or moment, and the anchor location.

The values of cable to chord length (straight line from one end of cable to another) ratios used in successful boom designs have varied between 1.06 and 1.25. The smaller value indicates that the arc of the boom cable is shorter

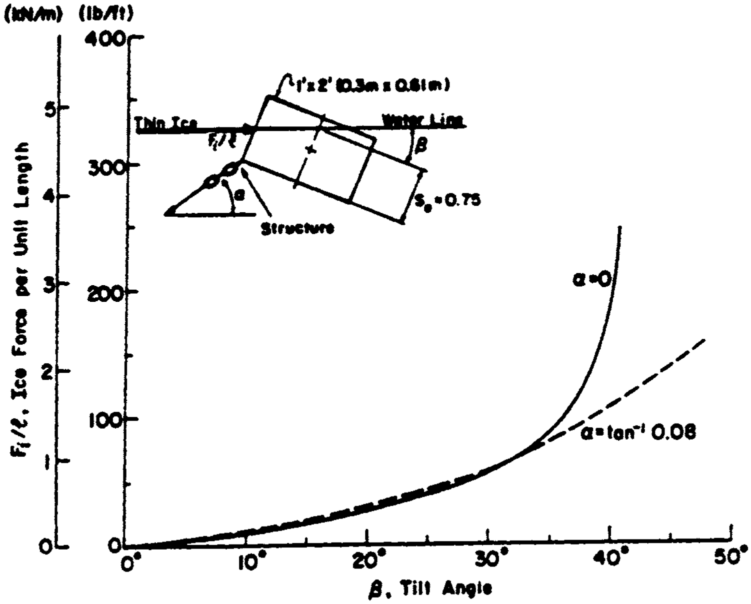


FIGURE 13-9. Ice Restraint Capability for a 1 × 2 ft Boom Timber for the Conditions Indicated.

so the cable develops greater tension. Occasionally a single boom cable reaching from shore to shore is used; however, the overall span is usually divided into several arcs of more or less equal length that are interconnected and individually anchored.

A failure analysis should be made of the multicomponent structure to estimate the increased loads exerted on members adjacent to a component that fails. The structure also should be evaluated for its response to a solid ice sheet that only acts at one or two locations along the boom and to an ice sheet that starts moving while frozen to the floating timbers. Under such circumstances the load increase may not be distributed uniformly to the anchor points. Figure 13-10 shows a timber ice boom in place. An example problem of ice boom design is contained in EM 1110-2-1612.

13.5.6 Floating Ice Dispersion

Ice problems at navigation locks are primarily caused by brash ice floating downstream or being pushed ahead of downbound traffic. The floating pieces of ice hinder gate opening and closing, stick to lock walls creating problems with vessel passage, and stick to lock gates causing operational problems. Large quantities of ice pushed ahead of a downbound ship can interfere with lock operation because a separate lock cycle solely for ice is often required by long ships using short locks. If ice could be prevented

FIGURE 13-10. Timber Ice Boom.

from entering the locks, most of these problems would not occur. A high flow, high velocity air screen was installed across the upper entry of the Poe Lock at the St. Marys Falls Canal at Sault Ste. Marie, MI. The screen created a high enough horizontal water velocity in the upstream direction to keep the downbound ice from being pushed ahead of traffic. This mechanism has been used more recently to keep debris out of lock approaches in Summer. Figure 13-11 shows installation of an air screen and a schematic of an air screen layout. Additional details of air screen design and an example problem can be found in EM-1110-2-1612.

13.5.7 Ice deflectors

Riverside docks can be protected from ice flows by concrete structures. Figure 13-12 shows two concrete piers used to protect a coal barge loading dock.

13.6 SOURCE

The majority of information in this chapter came from EM 1110-2-1612, *Ice Engineering*.

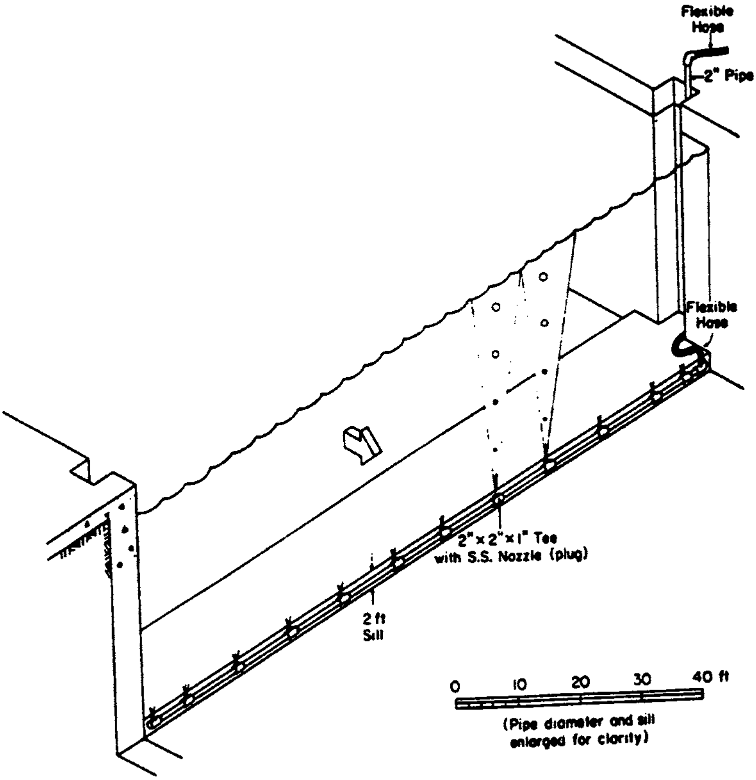


FIGURE 13-11. *Schematic of an Air Screen.*

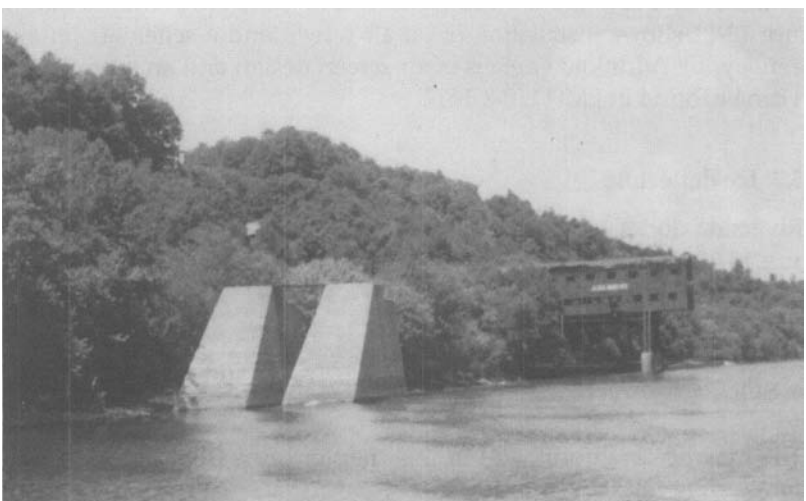


FIGURE 13-12. *Ice Deflectors on the Monagahela River.*

Chapter 14
ENVIRONMENTAL CONSIDERATIONS

14.1 EXISTING REGULATIONS

14.1.1 Background

With the passage and implementation of the National Environmental Policy Act (NEPA) of 1969 (Public Law (PL) 91-190), environmental impact assessments of water resource projects under the US Army Corps of Engineers and other Federal agencies assumed a greater level of importance. Previously, environmental assessments were controlled by internal regulations and were usually not distributed or reviewed outside the agency; subsequently, NEPA established a broad national policy directing Federal agencies to maintain and preserve environmental quality. Reference publications related to environmental aspects of navigation projects are provided in EM 1110-2-1202, *Environmental Engineering for Deep-Draft Navigation Projects*.

14.1.2 Environmental Impact Statement

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

14.2 FACTORS TO BE CONSIDERED

14.2.1 Background Environmental Considerations

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

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

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

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

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

14.2.1.5 Excavated Material. A major concern in many project areas will be the methods used to remove and treat excavated and dredged materials, depending on the nature of the materials and their potential for releasing contaminants.

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

14.2.1.7 Interruption of Migratory Routes. Evaluation of the use of the streams and adjacent terrestrial habitats as migration routes for aquatic and terrestrial animals is an important consideration during the planning process. Critical routes should be avoided when practical or provisions should be made for allowing alternate passage of the affected animals. Construction and/or maintenance activities could also be scheduled in such a manner as to avoid peak migration periods to reduce impact. Provision to pass migratory fish around or through the navigation structures are usually needed. Section 12.7 discusses passage of migratory salmon around locks and dams.

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

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

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

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

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

14.2.1.13 Ongoing Corps of Engineers Studies. Extensive studies on the upper Mississippi and Illinois waterways are evaluating the effect of navigation on fisheries, aquatic plants, mussels, shallow water habitat, water quality, recreational resources, and historic properties.

Chapter 15

COST ANALYSIS

15.1 COST OPTIMIZATION

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

15.2 ELEMENTS

The elements that are to be considered in an economic optimization or life cycle analysis are:

a. project economic life,
b. construction cost for various levels of protection,
c. maintenance costs for various levels of protection,
d. replacement costs for various levels of protection, and
e. benefits for various levels of protection using probability analysis.

15.3 EFFECTS OF PROTECTION LEVEL

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

15.4 ECONOMIC LIFE

A project economic life is generally 50 years; however, some projects such as cofferdams or temporary sheet-pile locks can have shorter project lives. Once the economic project life is selected the level of protection to design for is needed. This level of protection or condition to design for is related to the occurrence of physical events such as river discharge, wind speed, or ice thickness. The severity or magnitude of these events has a statistical distribution that can be ordered into a frequency of occurrence. The frequency is converted to exceedance probability and plotted against the level of protection as shown in Figure 15-1.

15.5 ANNUAL DAMAGE

The expected annual damages are computed using standard methods. The anticipated annual damages can be computed by multiplying the expected annual damages by the annual exceedance probability for various levels of protection. This anticipated annual damage value is added to the amortized construction cost, annual maintenance cost, and present worth

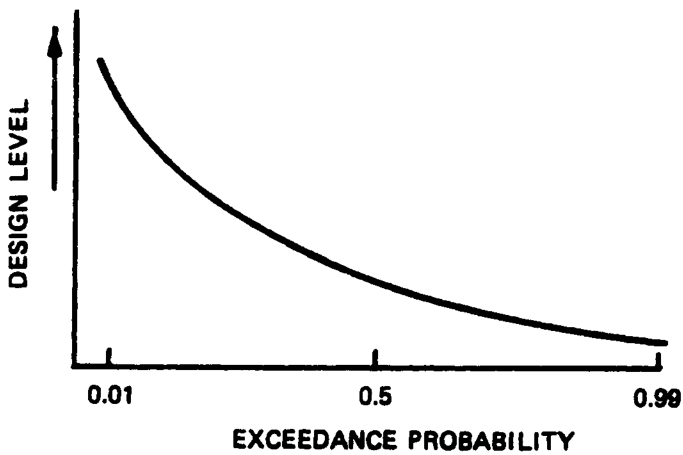


FIGURE 15-1. Probability Versus Protection Level.

amortized replacement cost to obtain the total project cost. A series of these total project cost estimates for various levels of protection will provide a total cost curve as shown in Figure 15-2. The optimum design is indicated by the lowest point on this curve.

15.6 TOTAL COST

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

15.6.1 Example Problem

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

a. 50 year project life.
b. 50 year life for concrete lock.
c. 25 year life for sheetpile lock.
d. Lost benefits during replacement due to construction of adjacent lock $2,000,000/year for 4 years.
e. Sheetpile filling time 20 min.; concrete lock filling time 8 min.
f. Average annual loss for slower filling is $1,500,000.
g. Interest rate 6%.

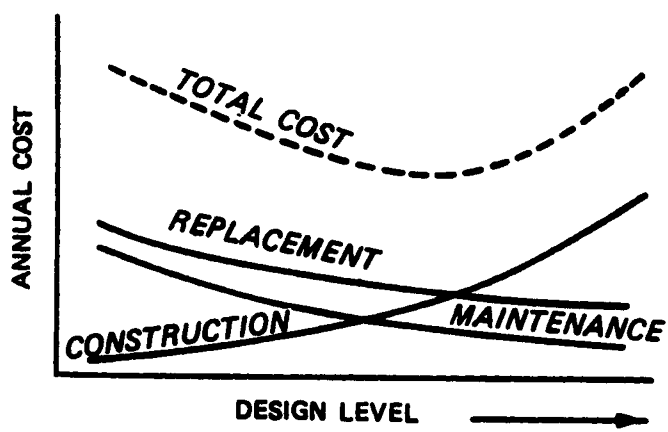


FIGURE 15-2. Project Cost Curves.

Find: Least annual cost lock using lifecycle analysis.
Analysis:

Step 1. Estimate initial construction cost.
Step 2. Compute present worth of replacement (using initial construction as equivalent dollar value for replacement).
Step 3. Estimate lost benefits incurred during construction of replacement lock.
Step 4. Compute present worth of lost benefits.
Step 5. Total present worth cost and amortize for project life.
Step 6. Estimate annual maintenance cost including lost navigation benefits during downtime.
Step 7. Estimate lost benefits for slower filling time lock.

Answer: The concrete lock has the least annual cost.
The project cost computations are presented in the following.

Concrete Lock	
Initial Cost	$60,000,000
Annual cost of construction (crf – 6% – 50 years)	3,806,400
Annual maintenance including lost benefits for downtime (50 year average)	50,000
Total Annual Cost	$ 3,856,400
Sheetpile Lock	
Initial Cost	$25,000,000
Replacement after 25 years (in today's dollar value) = $25,000,000	
Present worth (pwf′ – 6% – 25 years)	5,825,000
Present worth of loss during replacement construction ($8,000,000 – pwf′ – 6% – 25 years)	1,864,000
Total Present Worth	$32,689,000
Annual cost (crf – 6% – 50 years)	$ 2,073,800
Annual maintenance including lost benefits for downtime	500,000
Annual lost benefit for slower filling time	1,500,000
Total Annual Cost	$ 4,073,800

crf = Uniform annual series, capital recovery factor
pwf′ = Single payment, present worth factor

15.7 SPILLWAY OPTIMIZATION

An example of a spillway optimization study for a lock and dam on the Red River, LA is given in EM 1110-2-1605. This study varied the number of gate bays, adjacent overflow weir length, and levee raising cost. The study concluded that the alternative of a 6-bay spillway and 315 ft long overflow weir was the least expensive considering all the costs. The lock and dam structure cost of some of the alternatives were less than the selected plan, but their cost for additional flowage easement and levee raising caused their total cost to be higher.

This page intentionally left blank

Chapter 16
MODEL STUDIES

16.1 GENERAL

The development of satisfactory navigation conditions in the approaches to locks and dams and in critical reaches requires a knowledge of existing conditions with all navigable flows, changes produced by structures or modifications, and effects of changes on conditions affecting navigation. Adequate data are seldom, if ever, available to permit a reasonable analysis of the conditions existing in a particular reach, and time will usually not permit a detailed survey of the reach (some of the flows that should be considered might not be experienced for several years). Because of the complex nature of flow in natural streams, analytical studies to determine probable conditions from a particular plan of improvement are generally extremely difficult, even if sufficient field data on existing conditions are available. An example of how model studies modified an original design is shown in Figures 16-1 and 16-2. The modifications were needed to ensure safe approach conditions.

16.2 USE OF MODEL STUDIES

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

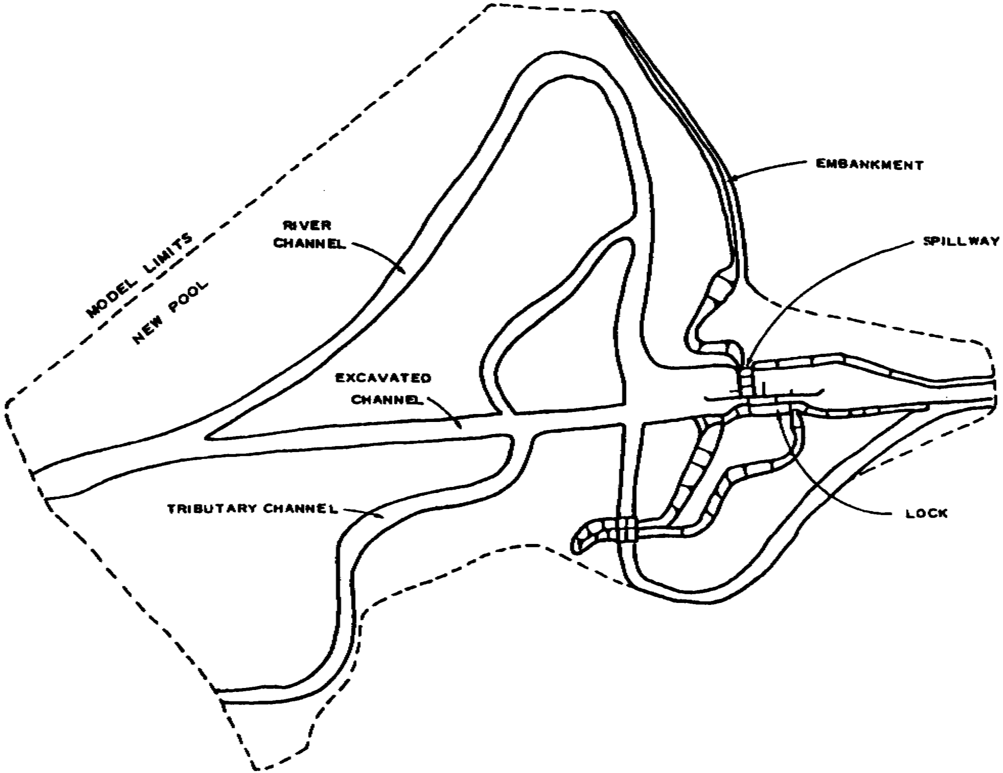


FIGURE 16-1. Original Plan for Lock and Dam.

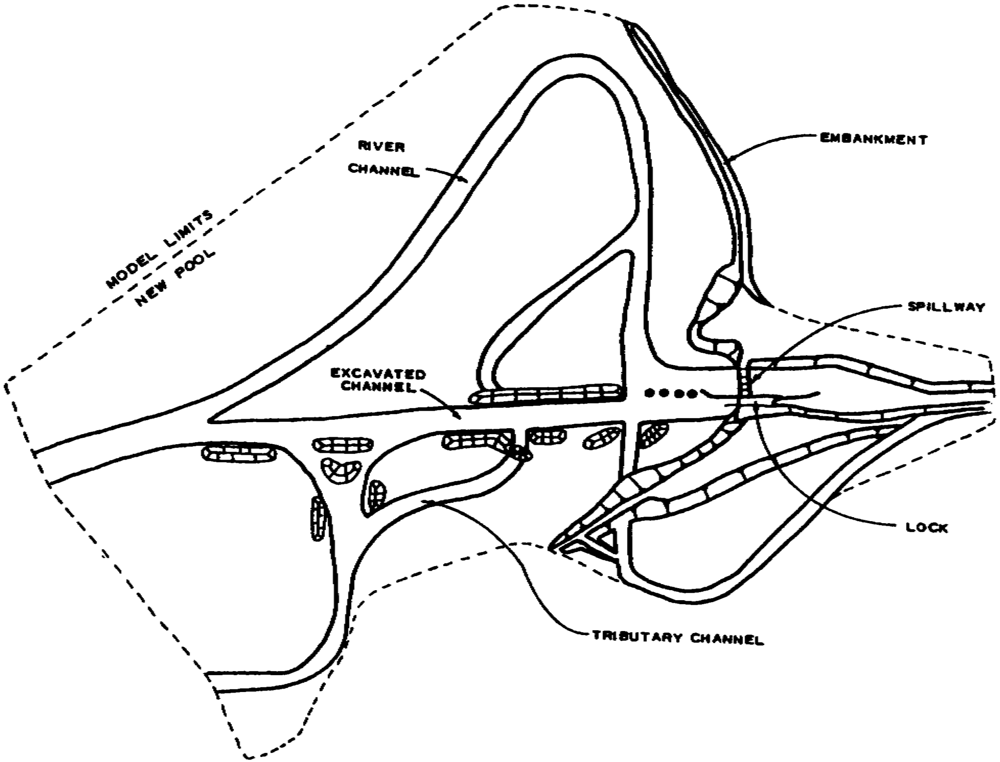


FIGURE 16-2. Improved Plan Based on Model Tests.

16.3 OPTIMUM DESIGN

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

16.4 COST OF MODEL STUDIES

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

16.5 SPILLWAY MODELS

Among the most common models for a navigation project is the verification of general spillway adequacy and performance. Generally, undistorted models of various linear scale ratios are used (commonly 1:12 to 1:60) depending upon the problems involved, and practical space and discharge limitations. A general model (see Figure 16-3) is normally used when approach conditions, flow over the spillway, and exit channel hydraulics are to be studied. A section model (see Figure 16-4) simulating one or more spillway gate bays is extremely effective for improving various details of spillway design at larger scales than the general model. If only a section model is to be used to simulate a structure, careful consideration should be given to

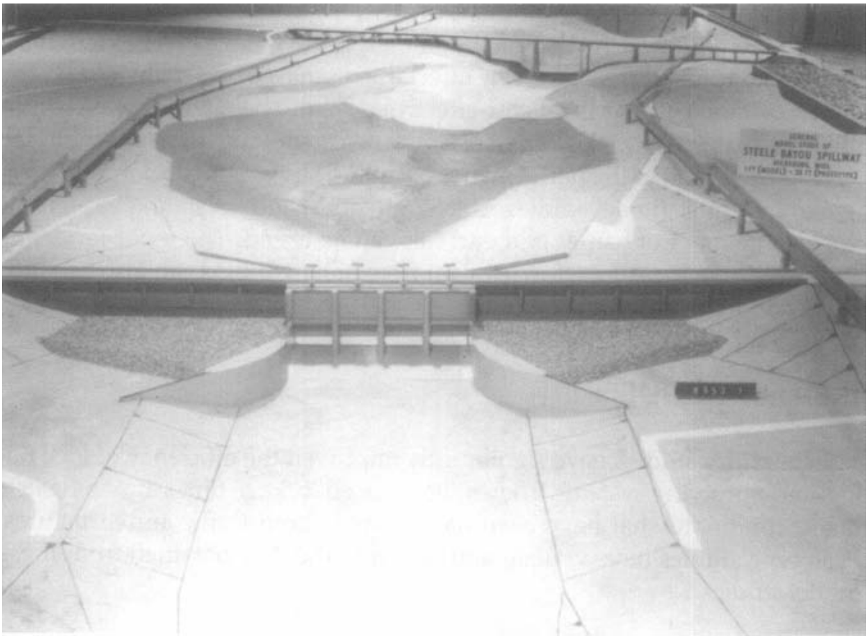


FIGURE 16-3. General Spillway Model.

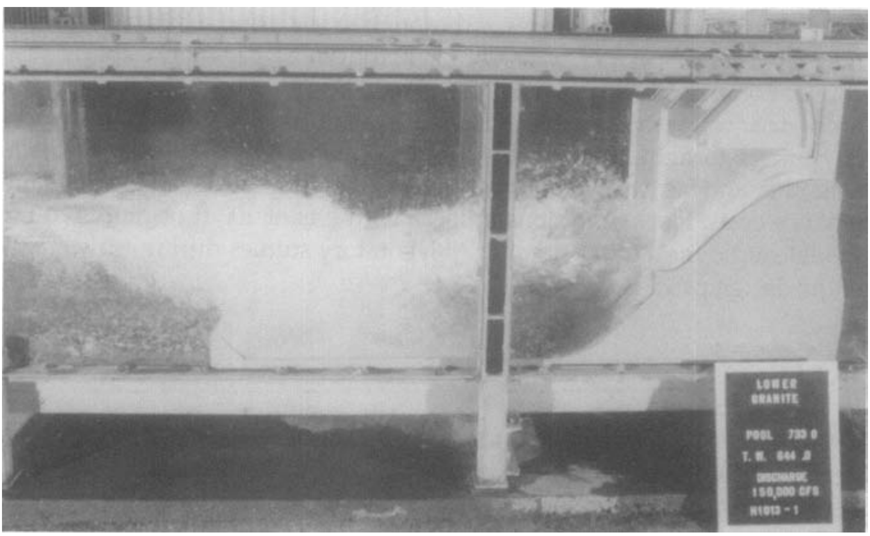


FIGURE 16-4. Section Spillway Model.

the model limits since a two-dimensional model may not introduce flow patterns that can be addressed in a three-dimensional model.

The effect of approach conditions on discharge of a navigation dam spillway and required excavation can be studied to advantage in a model. Abutment configuration may seriously affect the discharge of a spillway, and the model can indicate the most cost-effective design. The effect of waves from the ends of piers upon the height of sidewalls can best be studied in a model.

Determination of the performance of stilling basins is an important objective in hydraulic model studies. The length and width of stilling basins and the arrangement of baffles and end sills can be tested. The scour tendency and protective measures downstream from stilling basins can also be studied in a model.

16.6 LOCK MODELS

Laboratory studies have significantly improved the efficiency of lock filling-and-emptying systems. They have reduced lockage times and mitigated many conditions that have been hazardous to both traffic and structures. Prototype studies have verified and added to the data obtained from these model studies.

16.7 PURPOSE OF MODEL STUDY

Data for the design of a filling-and-emptying system for a low-lift lock are available. However, if the filling-and-emptying system under consideration varies from conventional types, a thorough study using a hydraulic model may be necessary. A lock with a lift of 40 ft or more generally departs from conventional designs, and normally cannot be confidently patterned after other designs. Even though problems are not apparent, a model study usually brings to light corrections or improvements in design that result in smoother and faster operation and effects savings in construction and maintenance costs. Flow conditions in locks with lifts of 100 ft or more require model studies and other specialized laboratory studies during early stages of the design process.

16.8 SCALES

A scale of 1:25 predominates for recent lock model studies. This scale ratio permits visual observations of turbulence and other flow conditions and permits the use of usual types of laboratory instruments for making measurements of pressures, velocities, discharges, and linear dimensions.

16.9 MODEL CONSTRUCTION

Construction materials used for lock models include metal and concrete; plastics are used for sections of conduit where observations of the interior flow conditions are desired and for forming curved surfaces such as entrances, bends, or dividing vanes. Where duplicate parts are required, such as lock chamber ports, lateral entrances, floor laterals, and the like, it has been found that accurate reproductions can be made in concrete by the use of wooden forms. Swelling or contraction, which are objectionable features of wood, is not experienced with concrete. Materials for the various parts of the model structure should be selected on the basis of their resistance to dimensional change, particularly those sections and surfaces that are exposed to flow or changing volumes of water. The new Bonneville Lock model is shown in Figure 16-5. Figure 16-6 shows the Bonneville Lock chamber as constructed.

16.10 PROTOTYPE EXPECTATIONS

A prototype lock filling-and-emptying system is normally more efficient than predicted by its model. The difference in efficiency is acceptable as far as most of the modeled quantities are concerned (hawser forces, for example) and can be accommodated empirically for others (filling time and overtravel, specifically). However, in circumstances in which knowledge of extreme pressures within the culverts in the prototype is important, additional corrections to the predictions from the model are required. These corrections are particularly important for high-lift locks in which questions regarding cavitation (resulting from extremely low pressures) are of concern.

16.11 REVISIONS TO SCALED VALUES

Adjustments to model-based coefficients for prototype application are based on one of the following three general approaches.

16.11.1 Filling-and-Emptying Times

General guidance is that the operation time with rapid valving should be reduced from the model values by about 10% for small locks (600 ft or less) with short culverts; about 15% for small locks with longer, more complex culvert systems; and about 20% for small locks (Lower Granite, for example) or large locks having extremely long culvert systems.

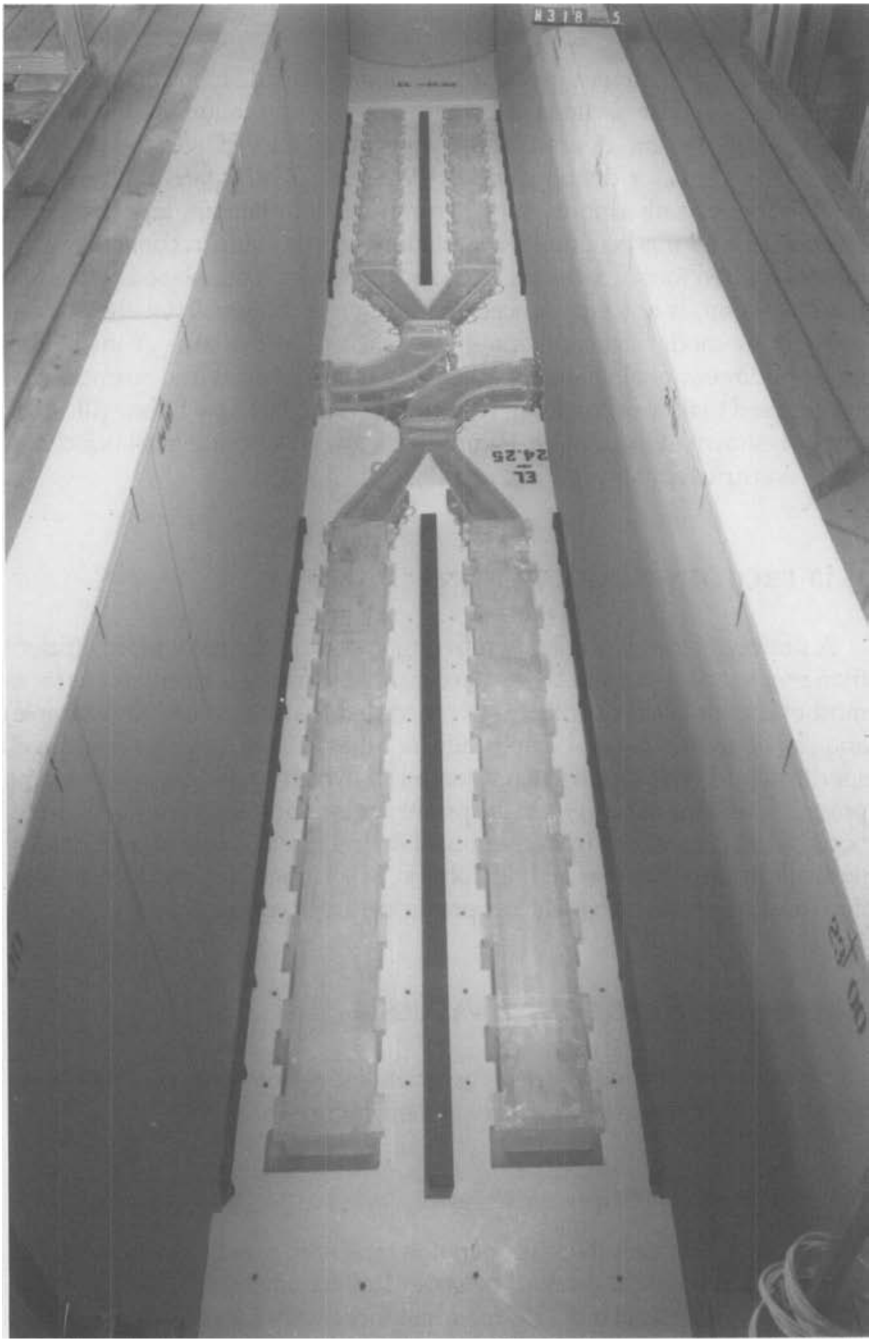


FIGURE 16-5. Fill-and-Empty Model for Bonneville Lock.

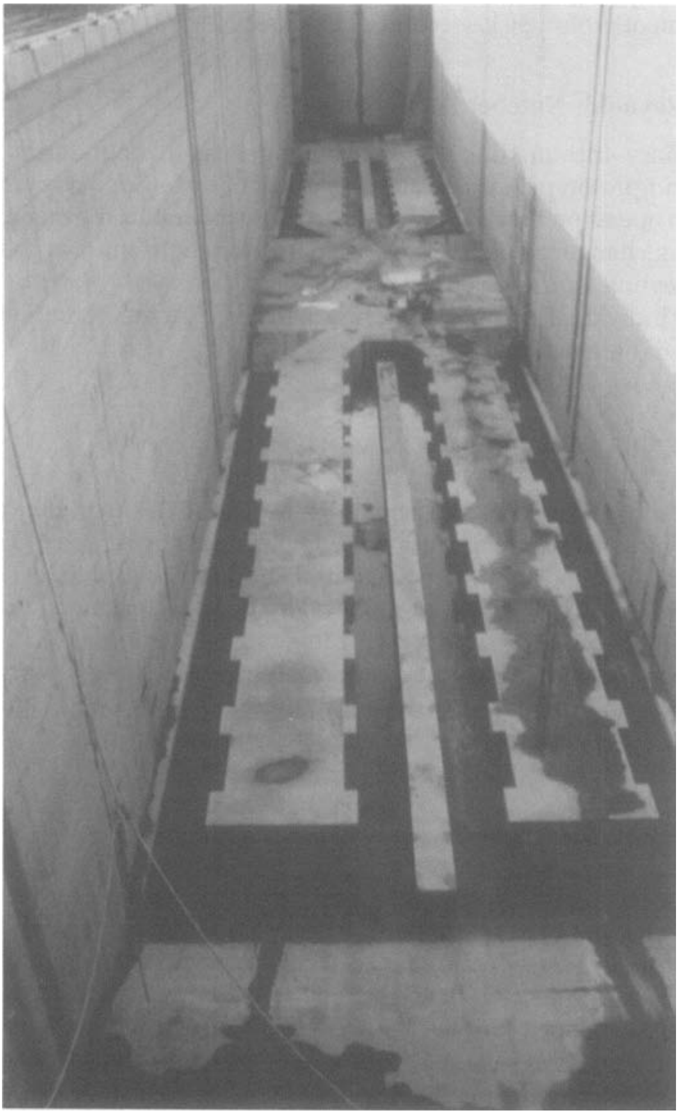


FIGURE 16-6. Bonneville Lock Chamber as Constructed.

16.11.2 Similar (Model and Prototype) Locks

A lock as similar as possible to the design lock and for which either operation time or culvert pressure data are available (model and prototype) provides a comparison such as in Section 16.11.1, or for pressure values, direct evaluation of prototype loss coefficient values.

16.11.3 Reynolds Number Corrections

Boundary friction differences, assuming smooth boundaries in both model and prototype, explain about one half of the efficiency change with regard to operation time for certain locks (Lower Granite, for example). The remaining change is due to undeterminable variations in form coefficients or the Reynolds number difference. Sensitivity analysis (systematic variations in individual form coefficients) permits extreme conditions to be accounted for in design.

16.12 GENERAL NAVIGATION MODELS

These models are similar to general spillway models except they are fixed bed and have sufficient upstream and downstream topography to evaluate approach conditions with remote-controlled model tows. Model scales are usually 1:70 to 1:120. Figure 16-7 shows the Bonneville general navigation model with a tow approaching the upstream guide wall. Figure 16-8 shows a time lapse of a downbound tow approaching the Bonneville Lock. Figure 16-9 shows an upbound tow departing the Bonneville Lock. Figure 16-10 shows the upper approach of the completed Bonneville Lock.

16.13 VESSEL SIMULATOR MODELS

A vessel simulator is a computer model that simulates the movement of a tow in a channel or lock approach. Simulators allow the use of both automatic controlled vessel navigation as well as navigation with human pilots with realistic tow controls in a simulated vessel bridge environment. A test scenario is developed for the channel or lock approach to include currents, wind, and structures. As the tow progresses through the test a computer-generated view from the pilot's window is displayed in color on a large screen (see Figure 16-11). This allows a pilot to observe the vessel in motion relative to physical objects that would be seen in the actual tow transit. Data such as tow motion, location, and orientation in the test area are recorded for later analysis.

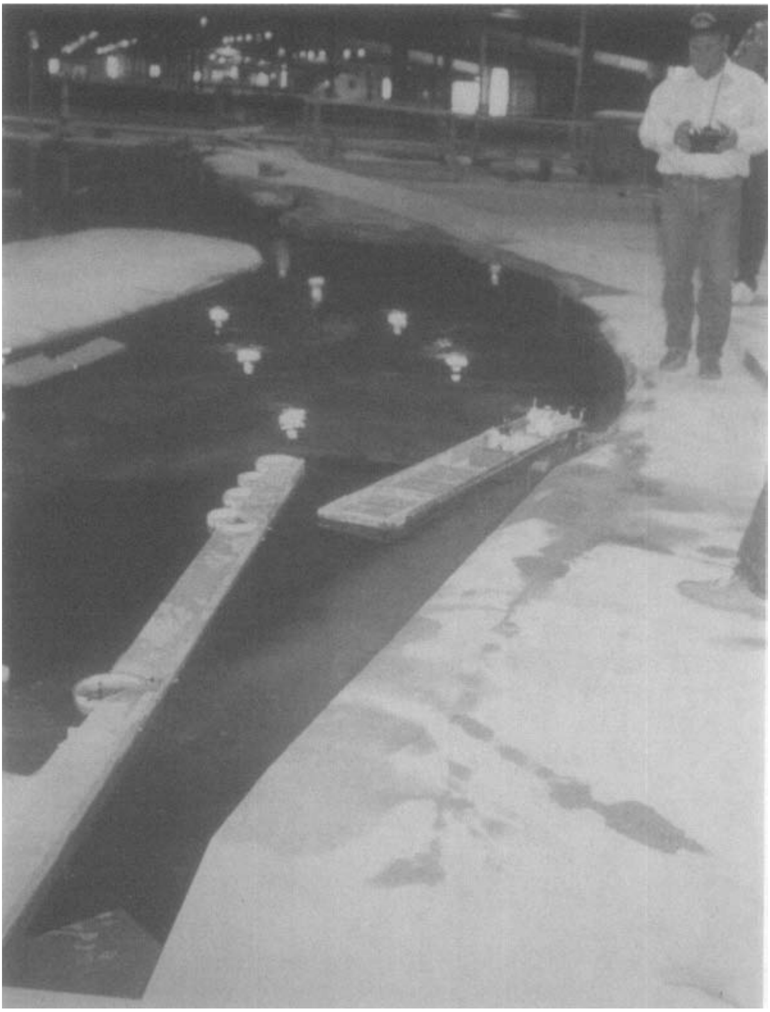


FIGURE 16-7. Bonneville General Navigation Model.

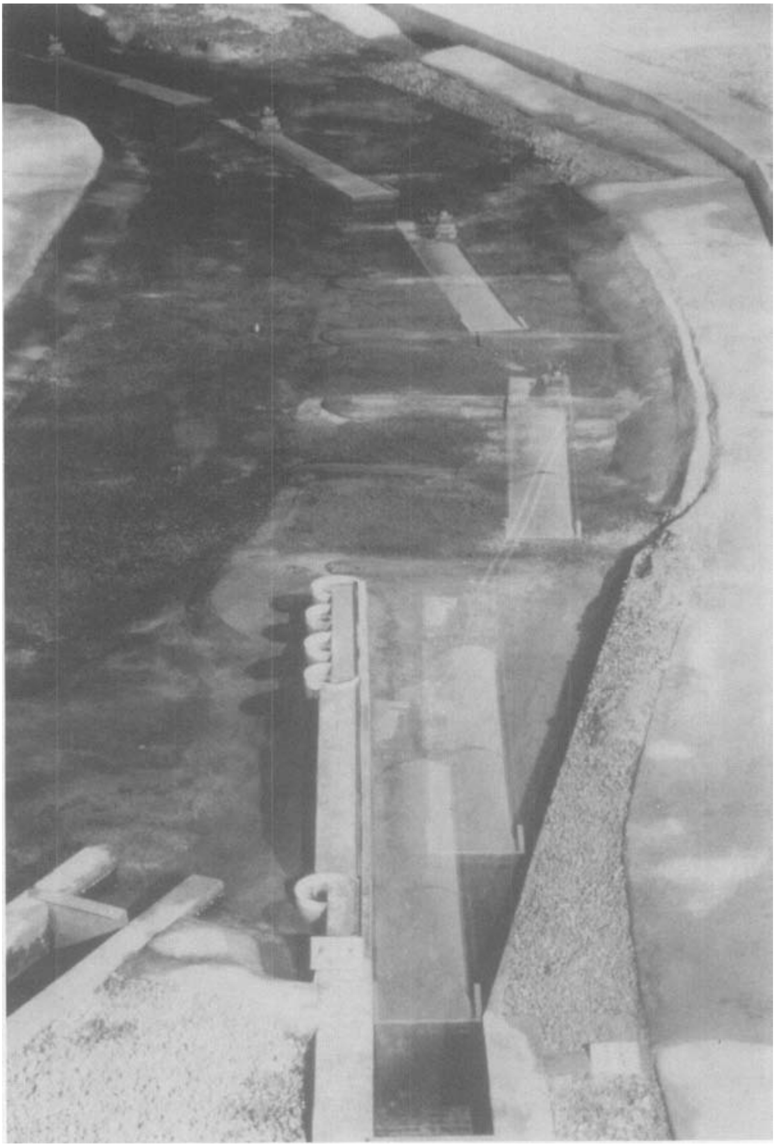


FIGURE 16-8. *Downbound Tow Approaching Bonneville Lock.*

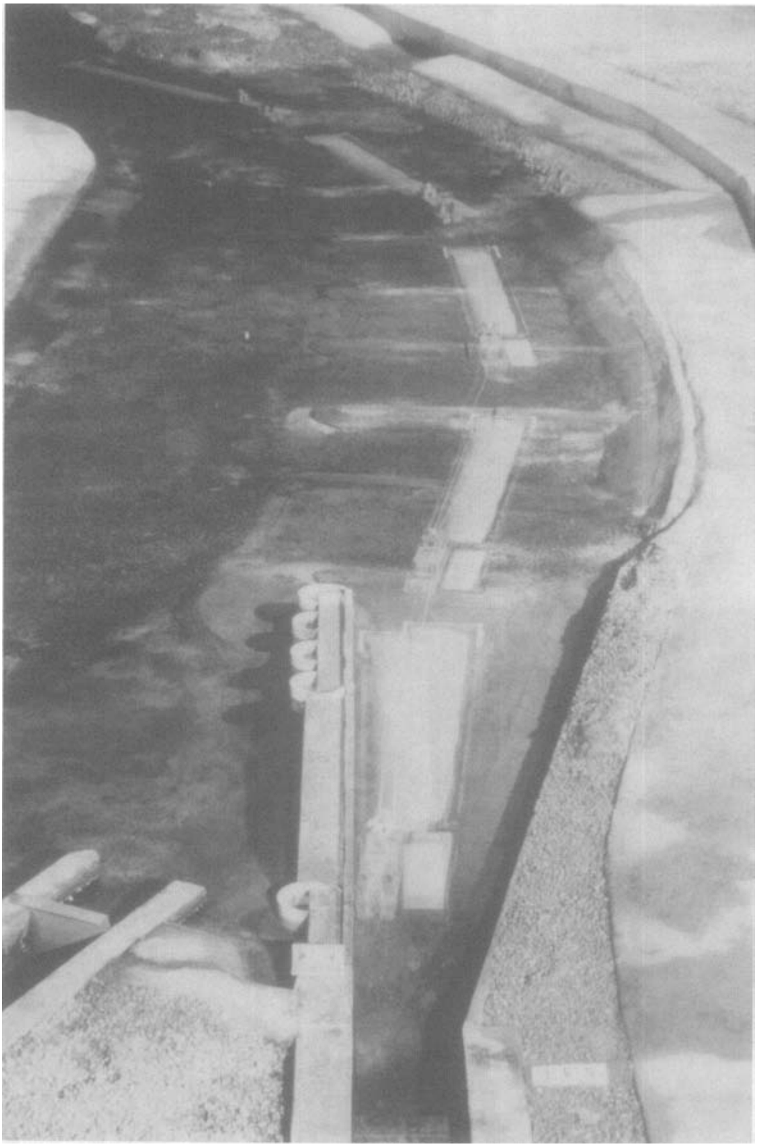


FIGURE 16-9. Upbound Tow Departing Bonneville Lock.

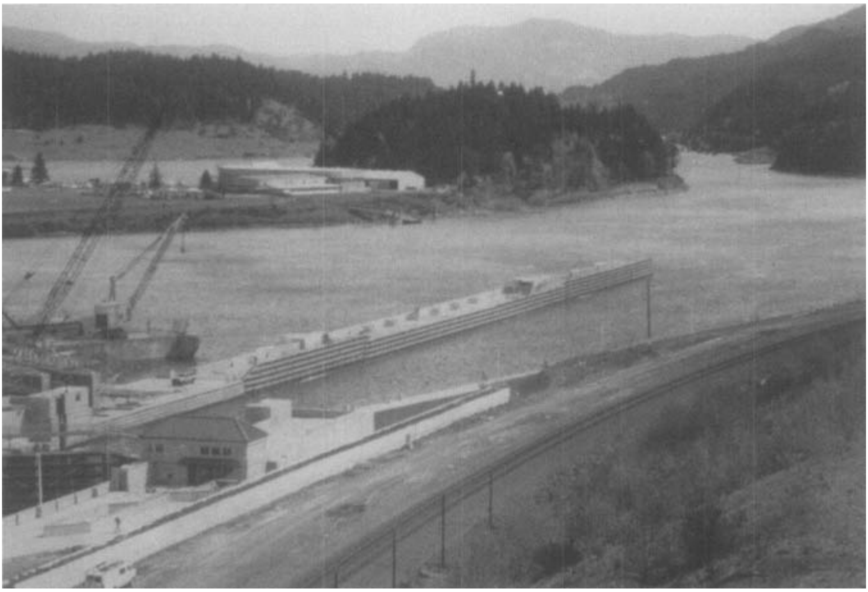


FIGURE 16-10. *Upper Approach of Completed Bonneville Lock.*

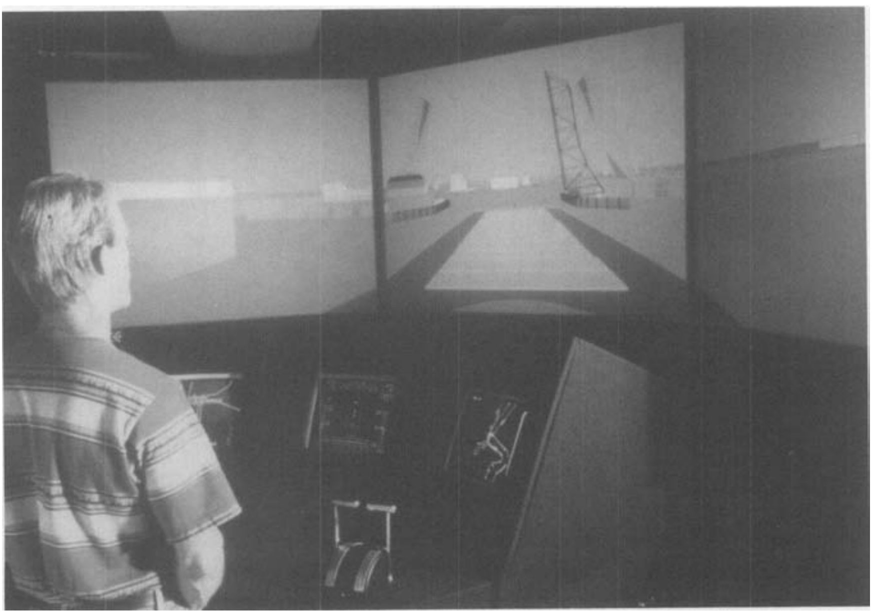


FIGURE 16-11. *Vessel Simulator.*

Chapter 17
CONSTRUCTION

17.1 OVERVIEW

17.1.1 General

Lock and dam construction can be in the existing river channel with cofferdams or in the dry. With "in the dry" construction, the river is diverted to the structure when construction is completed. A third alternative, not covered in this chapter, is float-in construction. This method has been used for hydropower structures and is presently being considered for lock structures.

17.2 IN-RIVER CONSTRUCTION

17.2.1 Factors to be Considered

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

17.2.2 Maintenance of Traffic

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

17.2.3 Effects on River Currents

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

17.2.4 Cofferdam Configuration

The scour along the riverward face of the cofferdam can be minimized by the use of a deflector. Rounded corners or deflectors designed to streamline flow will tend to reduce the depth of maximum scour but would maintain high velocities along the riverward face of the cofferdam. Deflectors can be designed to reduce or eliminate the high velocities along the main part of the cofferdam. Deflectors consisting of an upstream extension of the riverward arm of the cofferdam with a section angled about 45 degrees landward have been successful in containing the scour near the corner of the deflector and along the deflector itself, away from the main part of the cofferdam under pressure when dewatered (Figure 17-1). The length of the extension and the angled portion of the deflector would be based on the amount of contraction provided by the cofferdam and velocities of riverflow. The use of 150 to 200 ft upstream extensions

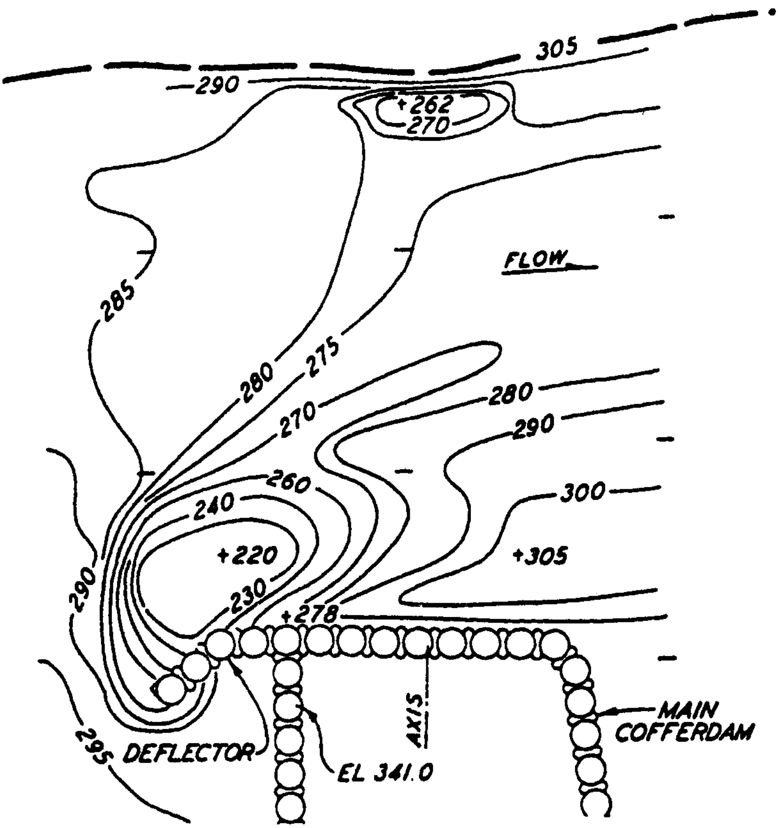


FIGURE 17-1. Typical Scour Pattern with Deflector.

with deflector arms at least that length has produced satisfactory results in tests of Mississippi and Ohio Rivers projects when the river channel was contracted as much as 50%. This type of deflector caused deposition along the riverward face of the cofferdam and moved downbound tows away from the cofferdam (Figure 17-2). The downstream arm of the cofferdam, extending normally or at an angle of not more than about 45 degrees in relation to the direction of flow, would generally be subjected to little or no scour since sediment moved along the riverward arm would tend to be deposited downstream of the cofferdam. Figure 17-3 shows the successful use of a cofferdam deflector during construction of the Melvin Price Lock and Dam.

17.2.5 Flow Diversion Schemes

Lock and dam construction normally requires a dry construction site. As these structures are usually located across or near streams, cofferdams are

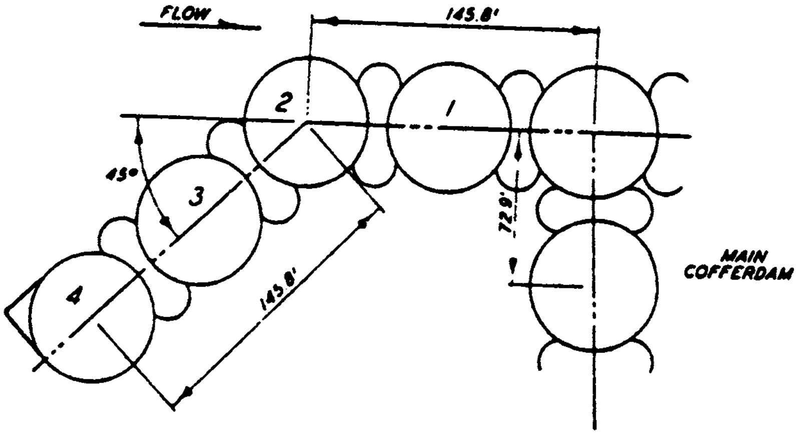


FIGURE 17-2. *Cofferdam Deflector.*

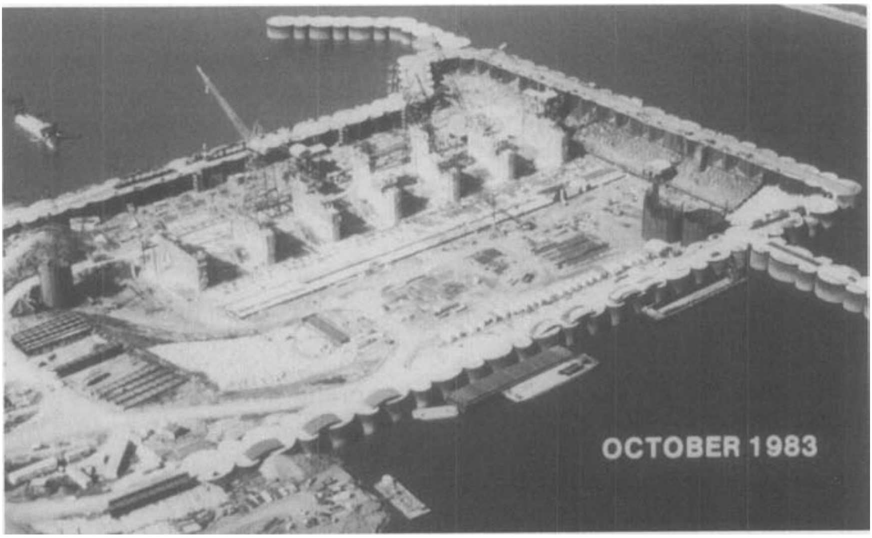


FIGURE 17-3. *Cofferdam Deflector During the Melvin Price Lock and Dam Construction.*

required for site dewatering and a reasonable degree of flood protection. The construction cofferdam usually creates a restriction in the river cross-section. Usually several alternate diversion schemes are investigated before the most feasible and economical solution is found. Several factors need to be considered in developing a diversion scheme.

17.2.5.1 Flooding. When designing a cofferdam scheme, an important design consideration is to limit upstream flooding to acceptable levels.

Although the flooding is only for the duration of construction, increased flooding may cause damage to agricultural, commercial, or other interests. An "acceptable" level depends on the general features and type of developments upstream from the construction site, cost of diversion structures, and cost of flooding the construction site.

17.2.5.2 Erosion. Another consideration is scour in erodible bed streams. Scour must not endanger the stability and/or constructibility of temporary structures (cofferdams) or create conditions that would differ substantially from design assumptions at the permanent structure. Deflector cells are sometimes constructed adjoining the upper arm of the cofferdam to direct flow away and thereby protect the main cofferdam. Scouring increases the cross-sectional area of the restriction and thus decreases the amount of induced upstream flooding. This may be taken into consideration during the cofferdam design. The stability of the riverbank at the restricted section must be analyzed. Temporary protection may have to be provided against induced erosive velocities.

17.2.6 Maintenance of Navigation

Diversion schemes should take into account that during construction, navigation may have to be maintained on the river. The restriction caused by the construction cofferdam must not create conditions hazardous to navigation by introducing currents that tows cannot negotiate. Temporary locks may be needed. A value of 4 mph (6 ft/sec) has been used to approximate velocities that tows can generally negotiate, although this depends to a great extent on the power of the towboat. Helper boats may be considered in some situations to assist underpowered tows. In addition to currents, towboats must be able to enter and leave the restricted section safely without damage to the structure. It is preferable to maintain an open navigation section as long as possible to minimize traffic delays.

However, at some construction sites this may not prove feasible, since the inclusion of even a relatively small portion of the dam in the first phase of the work may result in unacceptable navigation conditions. In this case, the construction sequence must usually begin with the lock so that it will be available for the passage of river traffic as soon as possible. In either case, special measures (reduced speed, helper boats, etc.) may have to be taken to ensure navigation safety. Alternatives of a navigation bypass channel, temporary lock, or portage system may be considered. In some cases navigation improvements can be constructed without interference to existing river traffic, by using a cut across a bendway. In this case, no special provisions for flow diversion are necessary. General hydraulic models with model towboats or navigation simulators are usually recommended for major navigation structures to evaluate various diversion schemes.

17.2.7 Construction Phases

Since an opening must be provided to divert riverflows and in some cases to maintain existing navigation, projects must be constructed in two, three, or more phases. In general, economy dictates as few construction phases as possible, because of the cost and time delay associated with removal and replacing of earth embankments or sheetpiling for cofferdam cells. However, the number of phases must be consistent with velocity limitations to prevent excessive scour and to maintain navigation. Also, savings in initial costs sometimes offset the disadvantage of time delay provided the project can be constructed within the generally adopted schedule. An example of this phase construction is the Melvin Price project that replaced Lock and Dam 26 on the Mississippi River near St. Louis. Figure 17-4 shows 6½ gatebays were constructed during the first phase. River traffic used the opening between the first phase and the Illinois bank during this phase. The second phase (Figure 17-5) involved the construction of the lock and the remaining one-half gatebay; during this phase the river traffic used the opening between the second phase cofferdam and the Illinois bank. Riverflows passed through the navigation opening between the second phase cofferdam and the Illinois bank and that portion of the spillway completed during the first phase. In the third phase, (Figure 17-6) the remaining gatebays and second lock were constructed and the first lock was available for river traffic.

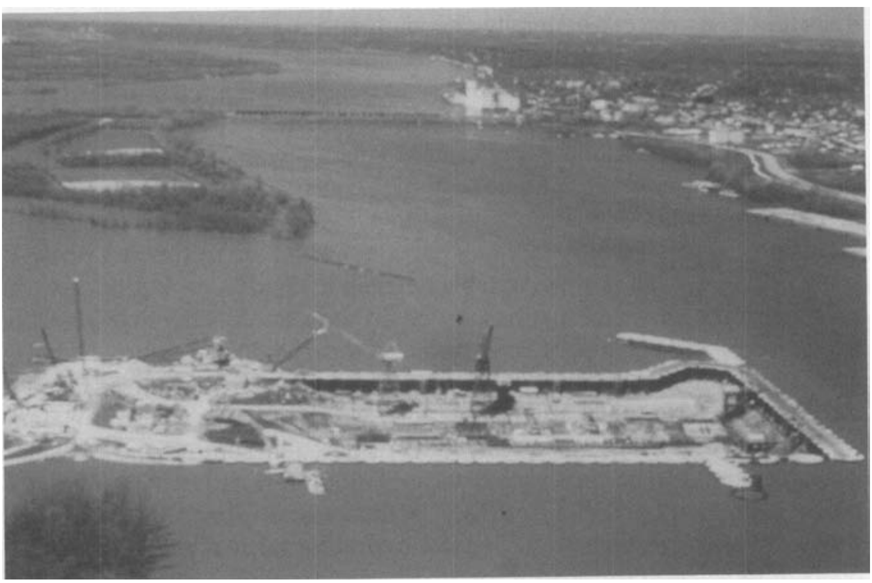


FIGURE 17-4. First Phase, Melvin Price Lock and Dam.

FIGURE 17-5. *Second Phase, Melvin Price Lock and Dam.*

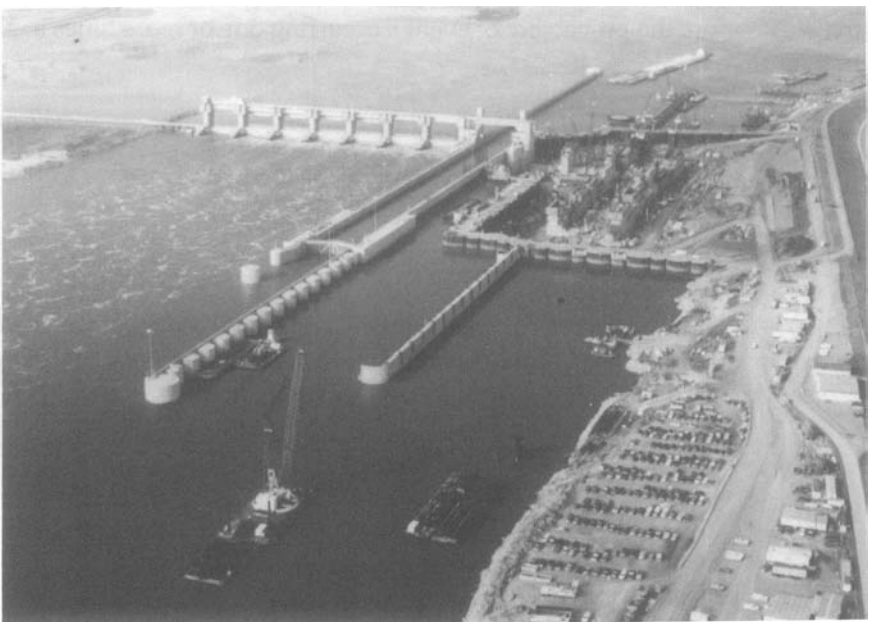


FIGURE 17-6. *Third Phase, Melvin Price Lock and Dam.*

17.2.8 Cofferdam Heights

Cofferdam layout and establishment of the cofferdam height are primarily oriented toward an economical plan to minimize hazards to construction activity, minimize costs of flooding on adjacent properties, and minimize costs of cofferdam construction. An economic analysis must be done for a range of cofferdam heights to find an optimum elevation. Factors that influence the decision include cofferdam cost for various heights, damage costs due to overtopping of the cofferdam by floods, costs due to delay in construction when the cofferdam is overtopped, risk of flooding during the anticipated construction period, cofferdam maintenance costs, the construction and diversion plan that is selected, and anticipated length of time required to complete construction. The determination of the probability of occurrence for the various frequency floods may be based on the formula:

$$P = \frac{N! p^i (1-p)^{N-i}}{i!(N-i)!} \tag{17-1}$$

where P is the probability of obtaining, in N trials, exactly i events having a probability p of occurring in a single trial. For the special case where $i = 0$, the formula becomes:

$$P = (1-p)^N \tag{17-2}$$

the probability of a flood event of magnitude p occurring zero times in N trials. Therefore the probability of event p occurring one or more times in N trials is:

$$P = 1 - (1-p)^N \tag{17-3}$$

For example, in a project with a 3 year construction period, $N = 3$. To analyze the flooding for a 10 year flood, $p = 0.1$. Therefore

$$P = 1 - (1 - 0.1)^3 = 0.271$$

or, a 27.1% chance that a 10 year flood will occur one or more times in a given 3 year period. The total probable flooding cost for each height of cofferdam can be computed by the formula:

$$C_t = P[(D)(C_1) + C_2] \tag{17-4}$$

where

C_t = probable total flooding cost
P = probability of flooding
D = number of days construction area is flooded before cleanup operation can begin

C_1 = investment losses per day while area is inaccessible
C_2 = fixed cost of cleanup.

17.2.9 Cofferdam Preflooding Facilities

When developing floods are so severe that cofferdam overtopping is predicted, scour damage and subsequent cleanup within the cofferdam can be minimized by preflooding the site. This can be accomplished by providing gated culverts or weir facilities with adequate capacity to raise the interior water level to near the river level prior to the time the river overtops the cofferdam. The photo in Figure 17-4 shows the Mississippi at flood stage with the water near the top of the cofferdam. Fortunately the river stage dropped and preflooding the cofferdam was not necessary.

17.2.10 Example Determination of Cofferdam Heights

An example of how to determine cofferdam height is presented in EM 1110-2-1605.

17.2.11 Scour Protection

Each construction scheme must be carefully analyzed to ensure that scour protection is provided where necessary. Successful protection has consisted of timber mattresses or riprap both with and without filter blankets, depending upon the soil types and flow conditions. Physical and numerical models have been useful in assisting the development of scour protection designs. The upstream riverward corner of the cofferdam is usually the critical point of scour potential. Wing extensions are sometimes added to the cofferdam to reduce velocity concentrations at this point.

17.3 CONSTRUCTION IN THE DRY

17.3.1 Benefits

Normally, construction in the dry is the fastest and least expensive construction method. Also the need for expensive cofferdams, phased construction, and traffic management during construction are eliminated.

17.3.2 New Bonneville Lock

The new Bonneville navigation lock is an example of dry construction. Figure 17-7 shows the preconstruction site with the powerhouse and old lock. Figure 17-8 shows the new lock built landward of the old lock. The only

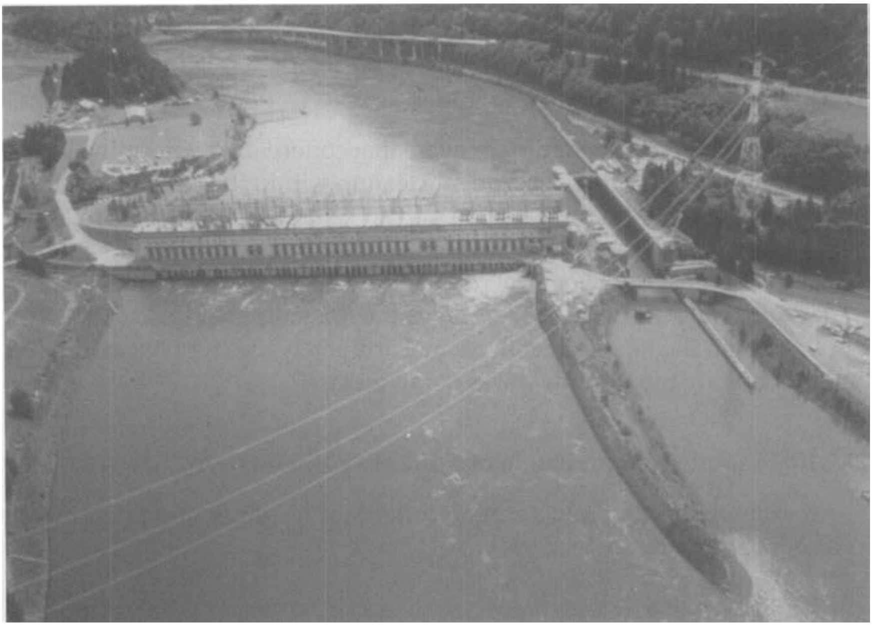


FIGURE 17-7. *Old Bonneville Lock and Powerhouse.*

FIGURE 17-8. *New Bonneville Lock.*

interference of vessel traffic was when the old lock guide wall was removed so the new lock wall could be constructed. During this period tugboats were used to assist tow movements into and out of the old lock.

17.3.3 Red River Waterway

Lock numbers 2, 3, 4, and 5 were also built in the dry. The Red River is a meandering river with many bends. This allowed lock sitting in the dry in a bendway. Figures 17-9 through 17-11 show Red River Lock and Dam construction in the dry.

FIGURE 17-9. Red River Lock and Dam Construction in Bendway Cutoff.

FIGURE 17-10. *Red River Lock and Dam Construction in the Dry.*

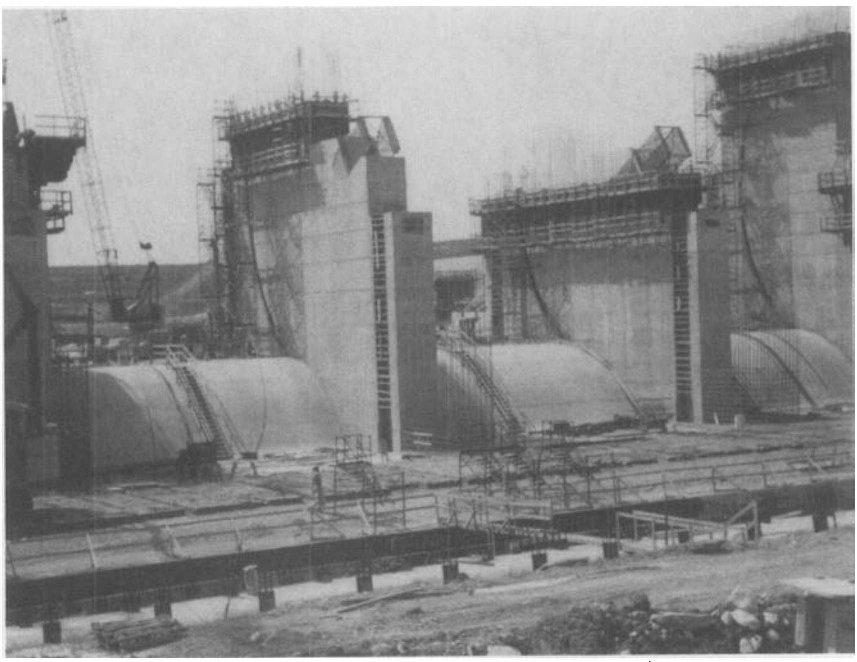


FIGURE 17-11. *Spillway and Stilling Basin.*

Chapter 18

OPERATION

18.1 NORMAL SPILLWAY OPERATIONS

18.1.1 Maintenance of Navigation Pool Levels

The purpose of maintaining a navigation pool on a river navigation project is to ensure that the authorized navigable depth is available all the time at every point in the river controlled by the project. In general, the point farthest upstream from the project, which would be the next navigation dam upstream in a system, or the "head of navigation" for a single dam, will be critical in this respect. The minimum pool elevation at which the preceding purpose is met is usually defined as the "normal pool."

18.1.1.1 Uncontrolled Spillways. These structures consist basically of a fixed-crest weir; a typical example is shown in Figure 18-1. The normal pool is defined as the upstream extension of the weir crest elevation for zero flow condition. The advantage of uncontrolled spillways is their simplicity of both operation and maintenance since the structure contains no moving parts (except for the locks) or equipment that could be subject to malfunctioning. The toe of the weir is subject to high-velocity turbulent flows and therefore requires relatively frequent inspection to preserve the integrity of the foundation. An operational disadvantage of navigation projects with uncontrolled spillways is the increased possibility of pleasure boat accidents. Since the drop in water surface at the weir is difficult to recognize from upstream, boats unfamiliar with the conditions may ram the weir instead of locking through. As riverflows increase, a pool elevation is reached where project navigation is suspended. In order to mitigate the effect of upstream flooding at uncontrolled spillways, locks are frequently used as floodways. Details of this special operation are described in EM 1110-2-1604.

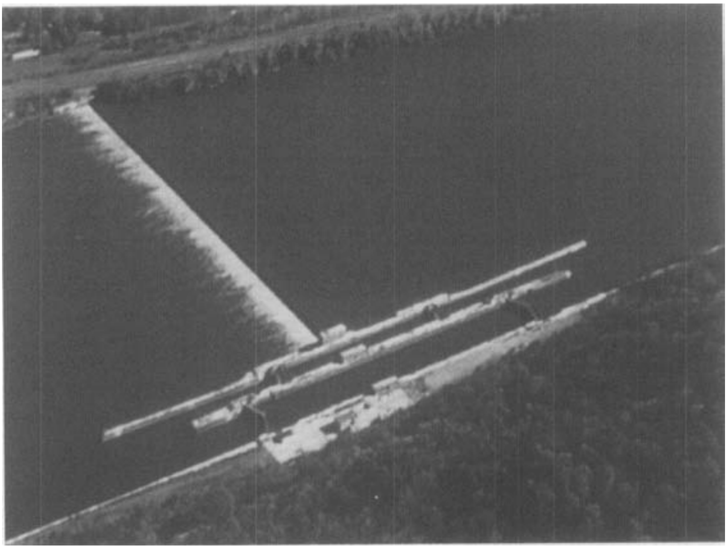


FIGURE 18-1. Uncontrolled Spillway.

18.1.1.2 Gated Spillways. The normal pool elevation is maintained by the operation of dam gates. It should be noted that in the case of multipurpose projects operated not only for navigation, other pool levels such as "minimum power pool" or "flood-control pool" may exist. These project operations are more complex than dams with navigation as their sole purpose. In the latter case, gates are operated as necessary to control all flows and to maintain a constant upper pool elevation (normal pool). At low dams, a normal pool is maintained until the tailwater reaches the normal pool elevation, at which time the gates are raised to maximum height and no further control of the pool level is possible. If the river level rises still farther, an elevation may be reached at which navigation is suspended and the project is prepared for flooding. A gate operation schedule should be prepared during the design stage. The schedule should be consistent with the design and should reflect any operational constraint imposed on the structure by the design. A frequent problem is scour below the spillway apron induced by misoperation of gates, especially at low tailwater levels. The operation schedule should minimize adverse impact on navigation at the upper and lower lock approaches. In general, this concept requires the uniform distribution of gate openings across the structure to prevent the formation of dangerous eddies downstream. Finally, the attainment of low operation costs and enhancement of water quality at low flows are also important operating objectives. In summary, from the operation standpoint, the gated structure offers greater flexibility to attain project objectives; however, the operation is more complex and requires a higher degree of maintenance to minimize equipment malfunction than projects with uncontrolled spillways. Also, the consequences of navigation accidents on project operation are likely to be more severe (loss of pool due to barges lodged under gates).

18.1.1.3 Movable Dams. At some locations, natural river discharges are sufficient during a portion of the navigation season (which could be continual throughout the calendar year, or extend over part of the calendar year only) to obtain the authorized navigation depth. This is an advantage from the operational standpoint since locking delays are eliminated. However, during periods of low discharges, the dam must be raised to ensure sufficient depth for navigation. Movable dams are structures that accomplish this objective. An early version of movable dams was the wicket dams on the Ohio River, the majority of which are now replaced by gated structures. The wicket is a narrow wooden leaf that when raised, is supported in an inclined position by a prop and when lowered, lies flat on the foundation just downstream of the sill. A large number of wickets side by side constitute a movable dam. The wickets are raised and lowered from a maneuver boat. The operation of the wicket dams in their original form is rather time-consuming and hazardous, especially during winter periods.

18.1.2 Low-Flow Periods

The operation of movable dams to ensure navigation depth during low-flow periods has been described in Section 18.1.1.3. No special operation procedures can be implemented at fixed-crest dams during low-flow periods; however, projects with gated spillways can be operated to improve water quality during these periods. A study conducted on the Ohio River found that dissolved oxygen content downstream of navigation dams during critical low-flow periods can be increased by concentrated gate openings. Before implementing such an operation, a careful check must be made to ensure that concentrated gate openings will not result in downstream scour, eddy action, and so on. A very special problem can arise in areas where during extremely low-flow periods sufficient water is not available for lockages. Provisions must be made for adequate storage under these conditions.

18.1.3 Flood Flow Periods.

18.1.3.1 High Dams. Navigation projects with high dams are usually constructed in areas where the topography and lack of dense development in the river valley permit the utilization of greater lift heights, sometimes in excess of 100 ft. An important distinguishing feature of these projects from the low dams is that the tailwater has no effect on the operation of most high dams. Usually the project is authorized to operate to satisfy the demands of navigation, hydropower, and possibly flood control. Flood control is normally achieved by spillway gate operation. However, the gates only control that portion of the flow that is not used for hydropower generation. During flood periods, spillway gates are operated to pass flood flow until extremely high discharges are reached that the gates no longer control. At this project, the lock walls are above the maximum high-water elevation, theoretically rendering navigation possible at all times. The minimum pool is established by providing for authorized navigation depth.

18.1.3.2 Low Dams. The operation of low dams during flood periods is controlled by both the tailwater and headwater. Spillway gates are raised for increasing spillway flows by maintaining the upper normal pool until the tailwater reaches that elevation. At this discharge, essentially open-river conditions exist and further increase in the riverflows cannot be controlled by project operation. If hydropower is part of the development, in contrast to high dams, power generation will be possible only during part of the year. Periods of flood flows are excluded due to insufficient head to operate the turbines.

18.1.3.3 Hinged Pool Operation. Under normal spillway operations, the gates are adjusted to maintain the established normal pool level at all times except when flood stages exceed the pool level at the dam. Then the gates are fully opened. Hinged pool operations, which are limited to flood flow periods, involve opening the gates in excess of that required to maintain the pool. Thus the lower reach of the navigation pool in the vicinity of the dam would be drawn down to below normal pool elevation. The amount of drawdown or "hinge" at the dam is controlled by the criterion of ensuring adequate navigation depth throughout the entire length of the pool. Three purposes for hinging pools and the consequences of doing so are described.

18.1.3.3.1 Stage Control. Purpose: The purpose is to provide navigation channel depth in the pool reach of the river for flows lower than a specified maximum discharge, at which the authorized navigation depth would exist naturally. In addition, control stage limits exist at a certain point or points within the pool that must not be exceeded for these ranges of flows. Thus, as discharges increase, approaching that specified maximum discharge, the pool at the dam must be lowered so stages at control point(s) upstream of the dam do not exceed the limiting stage.

Example: In the pool of Dam No. 26 on the Mississippi River, a 9 ft deep navigation channel must be maintained during flow periods of 210,000 cfs or less. In addition, stages at Grafton, IL, approximately 15 miles upstream of Dam No. 26 must not exceed 420.0 ft NGVD. During minimum flows, the pool level at the dam is maintained at 419.0 ft NGVD. As discharges increase, dam gates are opened further and the pool is drawn down so as not to exceed the limiting stage at Grafton, IL. When approaching a discharge of 210,000 cfs, the pool at the dam must be lowered to 414.0 ft NGVD to accomplish this purpose. When flows exceed 210,000 cfs, all gates are opened fully and open-river conditions exist. It can be seen that a "hinge" of 5 ft exists at the dam (419.0 to 414.0 ft NGVD) as discharges increase from minimum flows to those providing uncontrolled navigation depth.

18.1.3.3.2 Real Estate Acquisition. Purpose: For some projects, hinging the pool can reduce the required amount of flowage easement acquisition because of lowered postproject flow-line profiles throughout the pool.

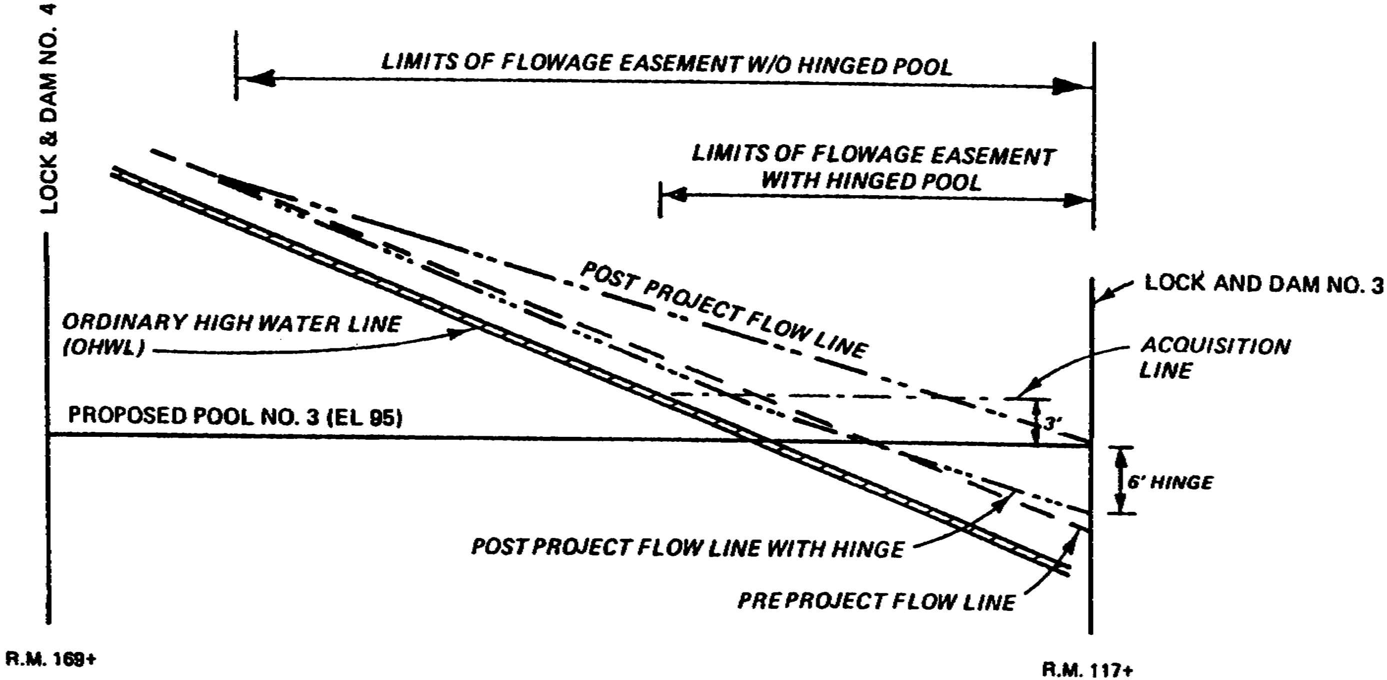
Example: For Pool No. 3 on the Red River Waterway Project, the criteria for real estate acquisition were the ordinary high-water line (OHWL) or the relationship of preproject versus postproject flow lines for any given discharge. Flowage easements were required where postproject flow lines were raised above both the OHWL and preproject flow lines for a given discharge. By hinging the pool, postproject flow lines can be depressed and the length of reach having flow lines above the OHWL can be reduced. Figure 18-2 illustrates the flow-line reductions that can be realized by hinging this pool.

18.1.3.3.3 Pool Dredging Quantities. Purpose: During the recession period of flood flows, sediment tends to be deposited in the middle portions of some pools. This occurs where the water-surface slope decreases because of the pool impoundment effects, and flow velocities are reduced. By hinging the pool, these deposits are carried farther downstream in the pool where post-flooding depths are adequate for navigation without dredging.

Example: Pool hinging to reduce dredging quantities has been tested in several pools on the Arkansas River navigation system. These tests indicated a potential for significant dredging reductions in some pools. Maximizing benefits requires a determination of the optimum time to initiate and to terminate the hinging process for each pool. Additional prototype testing in each pool would be required to optimize potential benefits.

18.1.3.3.4 Hinged Pool Consequences. If hinged pool operations are anticipated on a navigation pool, several factors must be considered in the project design.

1. The upper gate sill to the navigation lock must be set low enough so that navigation depths are provided while operating in the hinged pool mode.
2. Velocities and crosscurrents in the upper lock approach will be more severe than under normal navigation pool conditions.
3. Tie-up facilities along the lock guide and guard walls must be usable at the lowered pool levels.



NOTE: Q = 100,000 CFS FOR THIS ILLUSTRATION

FIGURE 18-2. Hinged Pool Operation.

4. Port, docking, and other facilities located within the affected portion of the pool need to be designed to avoid serious grounding problems from the lowered pool levels. Water withdrawal intake structures along the pool would also need to be designed to operate properly under lowered pool levels.
5. Sudden pool drawdowns can result in bank instabilities.
6. The increased complexity of operating the spillway gates for the hinged pool levels can lead to misoperations.

18.1.4 Ice and Debris Passage

18.1.4.1 General. A project operation plan needs to include methods of passing ice and debris. These methods can include both structural methods and operational procedures. See Chapter 13 for additional information.

18.1.4.2 Dam Gates. Regulating gates on a dam structure can be used to pass ice and debris either by underflow or overflow. In the first case, the gates are opened sufficiently wide to create enough flow that accumulated ice and debris are pulled from the upper pool to the lower pool, to be carried from the structure by the current. The magnitude of opening for successful operation depends on local conditions and experience; it is usually one third to fully opened gate depending on tailwater level. Hydraulic model tests give some indication of the required opening for new structures. One of the dangers of this operation is that scour holes downstream are often caused by this type of operation. To prevent occurrence of scour during ice or debris passage, the operation of the gates should not be in conflict with limitations established during the design phase. Floating ice and debris can also be removed by creating an overflow condition, whereby gates are lowered below the normal pool thus permitting the flow to carry the debris over the gate. Naturally, this "skimming" type of operation can only be accomplished on projects equipped with submergible gates. Also, this operation is ineffective against frozen-over ice conditions since it does not create enough drawdown to eliminate support for sheet ice as opposed to the method of opening gates described previously. At some projects on the Ohio and Mississippi Rivers, the use of submergible gates has been discontinued due to vibration problems. Both submergible tainter gates and roller gates are used successfully in the North Central Division of the Corps of Engineers on the Illinois Waterways and the Mississippi River, respectively.

18.1.4.3 Bulkheads. Some of the newer navigation structures are equipped with emergency gates or sectionalized emergency bulkheads. The primary design function of these structures is to protect against loss of the pool in emergency conditions caused by inoperative dam or lock miter gates. However, they can also be used for routine and nonroutine maintenance and to pass ice and debris. Usually, at least one of the bulkhead sections should be designed for overflow. This unit is placed second from the top in the assembled closure structure, which is then lowered to the closed position with the dam or lock gates closed. When the emergency closure is in place, the dam gates are opened, the top unit of the emergency closure is lifted, and ice and/ or debris is "skimmed" through the partially open emergency closure. As with the use of the gates, it is important to prevent scouring downstream of the structure.

18.1.4.4 Other Operations. In areas experiencing ice problems, common practice is to operate dam and lock gates to keep elements from freezing, even when not needed for river traffic or normal pool regulation. Seals on tainter gates are especially vulnerable to freezing. However, oil-heated seal plates have worked successfully at some projects. Ice also builds up between lower chord members of tainter gates and piers due to stilling basin turbulence. Often this is a greater problem than the seals.

18.2 SPECIAL SPILLWAY OPERATIONS

18.2.1 Purpose

Special spillway operations can be either intended or unintended. Intended operations may be due to such things as project repair, construction at the project or downstream, or grounded barges; unintended operation may be due to operator error, equipment failure, or tow impact with a dam.

18.2.2 Loss of Scour Protection

Failure of downstream stone protection below a stilling basin is an example of a condition that may require special operation. If the failure is localized below a limited section of spillway, reducing the opening of the spillway gates in that section or complete closing may be required until repair can be effected. Raising the tailwater elevation by operation of a downstream dam also may be effective in reducing the turbulence in the damaged areas. A combination may be required. Decreasing the flow in one part of a spillway will increase the unit discharge in other sections of a run-of-river project without storage available to adjust the spillway discharge. This can cause increased stress to undamaged sections of the stone protection.

18.2.3 Operator Error

Misoperation of spillway gates has the potential to create various problems with different degrees of seriousness. Outdrafts or adverse currents for navigation, or scour, can be created by the incorrect gate settings. Stone protection can be damaged or destroyed. Misoperation can cause abrupt changes in upper pool and tailwater elevations. It may also cause problems at adjacent locks or hydroelectric plants, such as inability to open lock lower miter gates due to a head differential across the miter gates. The changes in flows may cause problems, or require special operations, at upstream and downstream projects. The recovery operation must be executed so that abrupt changes in stage that could cause problems are not created.

18.2.4 Equipment Malfunction

Many types of equipment malfunctions may require special operations in order to recover normal capability. Some examples are covered in the following.

18.2.4.1 Jammed Gates. As in all cases, appropriate recovery procedures will depend on conditions and constraints existing at each given site. This may include placement of emergency closure in order to take the gate out of operation and adjustment of the remaining gate settings in order to compensate for the lost gate capacity. In general, it is important to correct the problem expeditiously in order to regain full operational capability and flexibility. It will be necessary for the emergency closure to be operable in flowing water.

18.2.4.2 Hoisting Machinery Breakdown. Appropriate recovery procedures in this case may begin with the attempt to close the crippled gate, if possible. If this can be accomplished, placement of emergency closure may not be necessary. If the gate load is not equally distributed on each side of the gate, the operator runs the risk of causing additional damage when attempting to lower the gate. If the gate cannot be lowered, it may be necessary to install the emergency closure.

18.2.4.3 Equipment Vibrations. Flow-induced vibrations have the potential for causing considerable damage to gates and other equipment. Vibrations can vary from the nuisance level to a major, structurally damaging problem. Appropriate immediate action may be to check the seals or sill for loose or jammed materials. Serious vibrations may require closing of spillway gates or other appropriate operational change in order to stop the vibrations until there is opportunity for evaluation and correction.

18.2.5 Spillway Maintenance

Limited gate availability operation occurs when one or more gate bays are closed for maintenance or repair work on the gates. The most important consideration in this operation is that the remaining gate capacity should be sufficient to handle anticipated high flows without causing increased upstream stages exceeding that predicted in the design. If feasible, repair and/or maintenance work should be scheduled during low-flow periods. On some projects, locks could be used as floodways should an emergency develop during repair work if they have been designed for this purpose.

18.2.6 Emergency Operation

18.2.6.1 General. All navigation projects need to develop a contingency plan for access to spillway gates so closure can be made in case of an accident. However, it will not be possible to include all possible conditions because each navigation accident will be different from others.

18.2.6.2 Navigation Equipment Collision with Spillway Gates and Piers. Potential for very serious damage to a navigation dam exists due to the presence of navigation traffic. Figure 18-3 illustrates an accident at Maxwell Lock and Dam on the Ohio River that occurred in December, 1985. In the case of collision, damage can vary from the inconsequential to major damage,

FIGURE 18-3. Accident at Maxwell Lock and Dam, Ohio River.

including loss of the navigational pool. Serious accidents are more likely to occur during high-water periods than during low water. For spillway gates, the two positions presenting the least potential for damage at many projects are in the fully raised position, particularly if this is higher than barges or tows passing through gate bays, and in the fully closed position. A particularly vulnerable position is with the gates slightly below or slightly above water level. In a rising river situation, with consequent increasing gate openings, it should be required operating procedure, as well as a design criterion, that the gates should be raised to a position above the highest expected water level or above a potential damaging level due to runaway tows or barges. Designers may find it prudent to include remote operating capability in order to permit quick action on the part of operators during emergencies.

18.2.6.3 Emergency Closure. Two types of closure devices are common.

18.2.6.3.1 Bulkheads. The most common type of emergency closure for spillway gate bays is a bulkhead consisting of one or more sections and commonly constructed of welded, high-strength, low-alloy steel. It contains two or more horizontal trusses with lateral and longitudinal cross-bracing and vertical tees between the chords of the trusses. A watertight skin plate generally provided on the upstream side, top and bottom seals, side seals, and roller assemblies complete the structure. The roller assemblies bear on bearing plates constructed in pier recesses. Usually several individual units are required to complete dam closure; some of these may be equipped with an overflow plate attached to the top truss. The purpose of such design is to utilize bulkheads for flushing ice and debris, when necessary. *The bulkheads should be designed for placement in flowing water.* Local geometry may make designs uncertain, so hydraulic model tests may be required to verify success. Most designs do not permit water flowing over and under the bulkhead units during lowering. Also, the stacking of more units may be required for successful placement on some projects. The units can be stored in a dogged position over the dam. In the latter case, an overhead gantry crane is used to transport the individual units to the gate to be closed. The first unit is dogged over the bay and the next unit is moved from storage, latched to the first one, and then the assembly is lowered and dogged a second time. Additional bulkhead units are latched to the assembly until complete closure is achieved.

18.2.6.3.2 Stop Logs. Stop logs usually consist of wooden beams that can be placed in the event of gate failure in recesses upstream of spillway gates. Generally, however, operating heads on the dam must be reduced before placement. Since this arrangement would result in partial or total loss of pool, they cannot be considered a true emergency closure.

18.3 MAINTENANCE DREDGING

18.3.1 General

The US Army Corps of Engineers maintains 25,000 miles of inland waterways that serve 400 ports and 130 of the nation's largest cities. The Corps dredges an annual average of 250 to 300 million cubic yards of sediment from this navigation system at a cost of about $400 million per year. Dredging is the single most costly item in the Corps Civil Works Budget.

18.3.2 Management Objectives

A dredging and disposal operation requires consideration of both short- and long-term management objectives. The primary short-term objective is to construct or maintain channels for navigation use. The long-term objective is managing disposal areas to maximize their longevity.

18.3.3 Dredging Operation Considerations

The following is a list of considerations for a dredging and disposal project.

a. Determine location and quantities to be dredged.
b. Characterize sediment composition.
c. Select proper dredging equipment.
d. Locate suitable disposal areas.
e. Control dredging operation to ensure environmental protection.
f. Manage containment areas to maximize storage capacity.
g. Develop long-term plan for maintenance dredging.

18.3.4 Location and Quantities to be Dredged

Most existing navigation channels will experience shoaling in specific locations. Periodic hydrographic surveys, inspection trips, or reports from tow boat operators will identify these shoaling areas. These areas could be at river crossings, upper ends of navigation pools formed by locks and dams, or downstream from tributary connections to the navigation channel. In most navigation projects with river training structures, shoal areas will recur at the same location. However, the volume to be dredged and dredging frequency will depend on the magnitude of floods that bring sediment into the navigation channel. Hydrographic surveys before and after dredging are needed to determine pay quantities and ensure navigation channel depths and widths are acceptable.

18.3.5 Sediment Composition

Sediment in shoals could be sand, silt, or gravel. Also the sediment could be contaminated. The sediment nature will affect the type of dredging equipment and disposal method. Sediment composition can be determined from soil samples or institutional memory from a recurring shoal site.

18.3.6 Dredge Equipment

The two most used types of dredge equipment are hydraulic pipe line and hopper dredges. The hydraulic pipe line dredge is normally used for shallow navigation channels (up to 20 ft deep) with little or no current or wave action. The hopper dredge is normally used in deeper navigation channels and can operate in moderate current and wave environments. EM 1110-2-5025 describes these dredges in more detail along with other less frequently used dredge plants. Figures 18-4 and 18-5 show a cutterhead pipeline dredge. Figure 18-6 shows a self-propelled hopperdredge.

18.3.7 Disposal Areas

An integral part of a maintenance dredging operation is to locate suitable disposal areas. The disposal areas should be confined or remote to prevent the dredging material from re-entering the navigation channel. An exception is agitation dredging which disturbs the sediment so it is resuspended in the water column and carried downstream by the current. Contaminated

FIGURE 18-4. Cutterhead Pipeline Dredge with Underwater View.

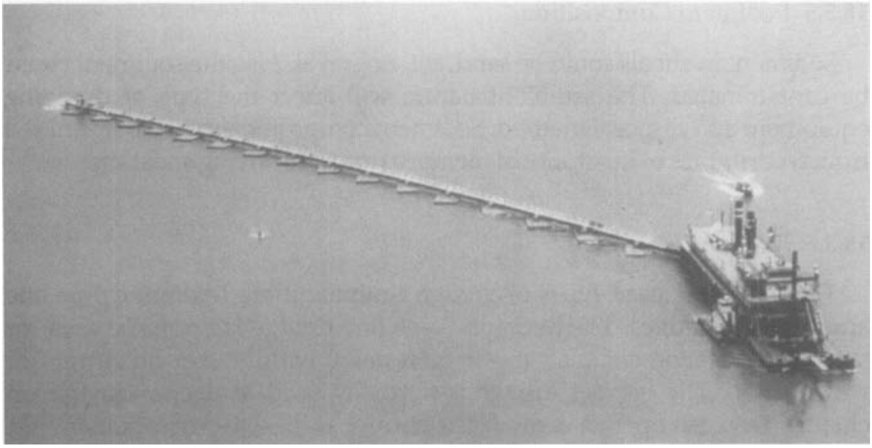


FIGURE 18-5. Cutterhead Pipeline Dredge Showing Pipeline Discharge.

FIGURE 18-6. Self-Propelled Hopper Dredge.

dredge material disposal should ensure confinement of the contaminates with no dispersal to adjacent land or water.

18.3.8 Environmental Protection

The dredging operation should minimize turbidity in the water column and proper dredging disposal of contaminated material.

18.3.9 Contaminated Disposal Area Management

Normally, disposal sites along a waterway are limited due to other land uses and protection of wetlands. Therefore disposal areas need to be managed to maximize their storage potential.

18.3.10 Long-Term Maintenance Dredging Plan

An operation and maintenance strategy for any navigation project should include a long-term maintenance dredging plan.

This plan should include:

a. periodic surveys,
b. estimate of dredging quantities annually and for the project life,
c. suitable dredging method,
d. adequate disposal sites,
e. environmental monitoring, and
f. periodic review of dredging volumes and disposal site capacity.

18.3.1 Additional Information

This is a brief overview of maintenance dredging for navigation projects. Additional information can be found in EM 1110-2-5025, *Dredging and Dredged Material Disposal* and other engineering manuals and publications by the US Army Corps of Engineers. Another information source is the US Army Waterways Experiment Station, (WES) in Vicksburg, MS. WES has been conducting research on dredging and dredge material disposal since 1970. Considering the large annual expenditure for this activity, the research on this subject is expected to be ongoing.

18.4 INSPECTIONS

18.4.1 Inspection

Periodic inspections of lock and dam structures are recommended to identify any structural or operational problems. These problems can then be scheduled for repair or replacement in a timely manner, so project function and safety are not jeopardized.

This page intentionally left blank

Chapter 19

REPAIR AND REPLACEMENT

19.1 GENERAL

Navigation dams will require major repairs, complete rehabilitation, or replacement when normal maintenance becomes excessive or structural integrity is threatened. Repair or rehabilitation is generally less expensive than replacement except where there are major structural stability problems.

19.2 DESIGN LIFE

The major rehabilitation goal is to extend the useful life of the project for 50 years. When a 50 year design life is not possible, a shorter design life can be recommended with suitable justification. Although the design life of most projects is 50 years, the practical usable life is much longer.

19.3 MODERNIZATION FEATURES

Modernization items should be considered in any rehabilitation plan. These items are intended to make the structure comparable to a state of the art replacement. Modernization items will be evaluated based on faster operating time, safety, reliability, and reduced manpower needs. Modernization items can include:

1. modern machinery,
2. modern electrical equipment,
3. remote controls,
4. television surveillance system including audio in some instances,
5. emergency closure, and
6. adding gates to ungated spillways.

19.4 TYPICAL REPAIR AND REHABILITATION ITEMS

The following are common items for major navigation dam rehabilitation projects.

19.4.1 Dam Stability

1. Replace upstream and downstream scour protection,
2. tendons through structure into foundation, and
3. cutoff of dam underseepage.

19.4.2 Discharge Capacity

1. Additional gates,
2. overflow dikes, and
3. raise dam.

19.4.3 Ice and Debris Control

1. Submerged gates,
2. control booms,
3. air screens, and
4. gate heaters.

19.4.4 Replacement in Kind

1. Resurface concrete surfaces,
2. repair or replace gates,
3. fix gate anchorages,
4. replace embedded metal, and
5. electrical and mechanical equipment.

19.5 SCOUR PROTECTION

19.5.1 Background

Inspections of the Corps of Engineers navigation dams (over 200) often show large scour holes downstream from the stilling basin. At some projects, the scour hole had undercut the stilling basin foundation to a point where remedial work was necessary. These scour holes are often caused by single gate operation to pass drift or ice during low tailwater conditions. Single gate operation produces jet flow that is constricted and intensified by return eddy currents in the stilling basin. Guidance for evaluation of major rehabilitation of existing projects follows.

19.5.2 Existing Project Design

Repair of existing projects requires evaluation of the same conditions as for a new project. However, remedial work is usually directed to the downstream protection because of the high cost of enlarging existing stilling basins. Design life of the remedial work can be based on judgment of how the original project performed. Hydraulic model studies are usually needed to verify the final design.

19.5.3 Consequence of Failure

An analysis of the failure consequences is needed. This would include repair and replacement costs and lost navigation benefits as well as loss of life and property. Very conservative design conditions are usually selected for a project on a busy waterway with sizable downstream population.

19.5.4 Design Rationale

This guidance must be site-adapted to specific project conditions. The design engineer is responsible for developing a safe, efficient, reliable, and least-cost plan with adequate consideration of environmental and social impact. Design innovations based upon sound judgment that are well documented are encouraged.

19.5.5 Fixed-Crest Dams

Scour downstream from fixed-crest dams is often caused by high velocity and excessive turbulence exiting the spillway apron. Modifications to the existing dam are often required before a suitable scour protection plan can be implemented. If there is evidence of piping of underlying materials through the stone protection, the cause may be fluctuating pressures or excessive ground water pressure. The repair should consider appropriate filters.

19.5.6 Gated Structures

Gated structures usually have a stilling basin that dissipates energy adequately when the project operation schedule is not violated. Scour downstream from these structures is usually caused when the gates are misoperated due to ice or debris passage and occasionally navigation accidents. A typical example would be a single gate that is raised higher than the operation schedule allows in order to pass ice through the structure. Generally during periods when ice passage is required, the tailwater is very low or at minimum elevation. The increased discharge due to the gate being raised higher than normal and the low tailwater cause significant turbulence in the downstream channel oftentimes resulting in severe scour and failure of the stone protection. Another flow condition that causes scour downstream from a gated structure is an undulating jet. This occurs when high tailwaters force the flow entering the basin to undulate and ride the surface of the tailwater through the basin and then plunge through the tailwater after leaving the basin. The plunging jet oftentimes is strong enough to reach the streambed or the stone protection and cause scour.

19.5.7 Methods of Protection

Some Corps districts have already begun to repair the scoured areas below navigation dams using graded stone protection and grout-filled bags. Site-specific model studies are oftentimes used to select an appropriate scour protection plan. Graded stone protection has been used by the St. Paul District on many of their navigation projects located on the upper Mississippi River. Model studies on some of these projects revealed that if the existing scour holes were armored with a large graded stone the structure could be protected. Grout-filled bags were used by the Pittsburgh District at Emsworth Dam on the Ohio River. The bags were used as an emergency replacement for large rock that probably failed during ice passage. Sunken barges filled with grouted rock are being considered for scour repair at Dam 2 on the Arkansas River. This repair method has the advantage of being able to be placed in the wet.

19.6 REPAIR AND REHABILITATION MODEL STUDIES

The following model studies for major rehabilitation have been conducted to address repairs.

Repair and Rehabilitation Model Studies

Project	Feature	Problem	Recommendation
Arkansas River Dams	Spillway gates	Gate vibrations	Remove seals on the bottom of the gate. Projects requiring bottom gate seals should use Type D configuration shown EM 1110-2-1605, Figure 5-19
Cheatham Dam	Spillway gates	Modify partially submergible gates to lift gates	Retain original gates and modify the sill and trajectory (Add 1.2 ft to sill elevation and an $x^2 = 26.8y$ trajectory over the original 1-on-1 slope)
Upper Miss. River Locks No. 2-10	Scour repair downstream Stilling basin Gated structures	Excessive scour during past 40 years of operation	Provide additional scour protection by underwater placement of quarrystone and graded riprap as determined in model tests
Montgomery Dam, Ohio River	Scour repair downstream Stilling basin Gated Structure	Excessive scour	Provide better toe protection and filter
Emsworth Dam, Ohio River	Scour repair downstream Stilling basin Gated structure	Excessive scour	Provide protection with large riprap or grout-filled bags
Allegheny, Ohio, and Monongahela Rivers	Scour downstream from stilling basin or structure Uncontrolled structures	Excessive scour	Provide protection with large riprap, grout-filled bags, sunken barges filled with grouted riprap, and/or modify structure
Dashields	Scour repair Uncontrolled structure	Excessive scour	Provide protection with large riprap and modify stilling basin
Pike Island, Ohio River	Scour repair Gated structure	Excessive scour	Provide protection with large riprap
L&D No. 2 Arkansas River	Scour repair Gated structure	Excessive scour due to barge accident and low tailwater	Sunken barges filled with grouted riprap

This page intentionally left blank

Chapter 20

CASE HISTORIES

Five recent Lock and Dam projects and one 1870s vintage project are presented to illustrate the application of the guidance given in this Manual. The unique features of these projects are briefly summarized here, with more detail provided in the remainder of the chapter.

- *Case History 1. Tennessee–Tombigbee Waterway.* The 236 mile long waterway connects the Tennessee River to Mobile Bay. There are 10 locks on the waterway.
- *Case History 2. New Bonneville Lock.* This lock replaces a smaller lock at the Bonneville Dam site on the Columbia River. The dam is 40 miles upstream from Portland, Oregon.
- *Case History 3. Melvin Price Lock and Dam.* This project is two miles downstream from the old Lock and Dam 26 on the Mississippi River near St. Louis, Missouri.
- *Case History 4. Red River Waterway.*
- *Case History 5. Leland Bowman Lock.* This lock replaces the Vermilion Lock on the Gulf Intercoastal Waterway in Louisiana.
- *Case History 6. Willamette Falls Locks.* This lock was built in 1872 and is still operating. It represents early navigation lock design that evolved into the modern system.

CASE HISTORY 1. TENNESSEE–TOMBIGBEE WATERWAY

Description

The Tennessee–Tombigbee Waterway was first proposed when the route was in the Louisiana Territory before acquisition by the US. The project is in both Alabama and Mississippi and connects the Tennessee River to Mobile

Bay. The 236 mile long waterway is divided into 3 major segments. The lower reach follows the Tombigbee River route and has 4 locks. The midreach is in a canal adjacent to the Tombigbee River and has 5 locks. The upstream reach is a land-cut canal through the divide separating the Tombigbee watershed from the Tennessee watershed. This divide-cut reach has one lock at the downstream end. Figures 20-1 through 20-4 are photographs of this project.

Pertinent Data

Lock Usable Length	600 ft
Lock Width	110 ft
Design Tow Dimension	600 ft × 105 ft
Design Tow Draft	9 ft

Lock Lift and Fill System Type

Lock	Normal Lift (ft)	Fill System
Gainesville	36	bottom longitudinal
Aliceville	27	side port
Columbus	27	"
Aberdeen	27	"
A	30	"
B	25	"
C	25	"
D	30	"
E	30	"
Bay Springs	84	bottom longitudinal

Construction Chronology

	Start	Open to Navigation	Construction Cost ($ millions)
Gainesville Lock	Nov. 1972	Oct. 1978	$32.3
Bevill Lock	Mar. 1974	Dec. 1979	$45.0
Columbus Lock	Apr. 1975	Jan. 1985	$44.7
Aberdeen Lock	Apr. 1975	Jan. 1985	$43.3
Lock A	Mar.1977	Jan. 1985	$23.3
Lock B	Jun. 1978	Jan. 1985	$33.5

FIGURE 20-1. Tennessee–Tombigbee Waterway, Columbus Lock and Dam: River Section.

FIGURE 20-2. Tennessee–Tombigbee Waterway, Lock and Dam A: Canal Section.

FIGURE 20-3. *Tennessee–Tombigbee Waterway, Bay Springs Lock.*

FIGURE 20-4. *Tennessee–Tombigbee Waterway, Divide Cut.*

	Start	Open to Navigation	Construction Cost ($ millions)
Lock C	Oct. 1978	Jan. 1985	$28.3
Lock D	Dec. 1980	Jan. 1985	$43.9
Lock E	Apr. 1981	Jan. 1985	$47.3
Bay Springs Lock	Apr. 1979	Jan. 1985	$74.4
Divide Cut	Apr. 1974	Jan. 1985	$486.0

Project Cost

Total project cost as of September, 1994 was $1.991 billion. This cost includes design, construction management, real estate, relocation, new bridges, channel modification, and new channels (307 million cubic yd excavated); recreation facilities (40 recreation areas), and lock and dam construction.

Unique Features

- Bay Springs Lock and Dam does not have a spillway. The Bay Springs Reservoir is controlled by the Pickwick Spillway on the Tennessee River.
- The 31.7 mile long divide cut has a navigation channel 12 ft deep and a 280 ft bottom width. The average cut is 50 ft and depth to the ridgeline is 175 ft. Rock revetments protect both sides of the channel.
- The Canal section was designed with a levee on only one side. The other side was left open to inundation for creation of wildlife habitat. This plan saved construction cost and created an excellent sport fishery.
- Project was completed 1½ years ahead of schedule and lower than estimated cost.

CASE HISTORY 2. NEW BONNEVILLE LOCK

Description

The new lock at the Bonneville Dam project is located on the Columbia River about 40 miles east of Portland, OR. The lock dimensions match the 7 upstream locks on the Columbia–Snake River navigation system. This lock replaces the original (1938) lock which was the first lock on the system with dimensions of 76 ft wide and 500 ft long. Figures 20-5 through 20-10 are photographs of this project.

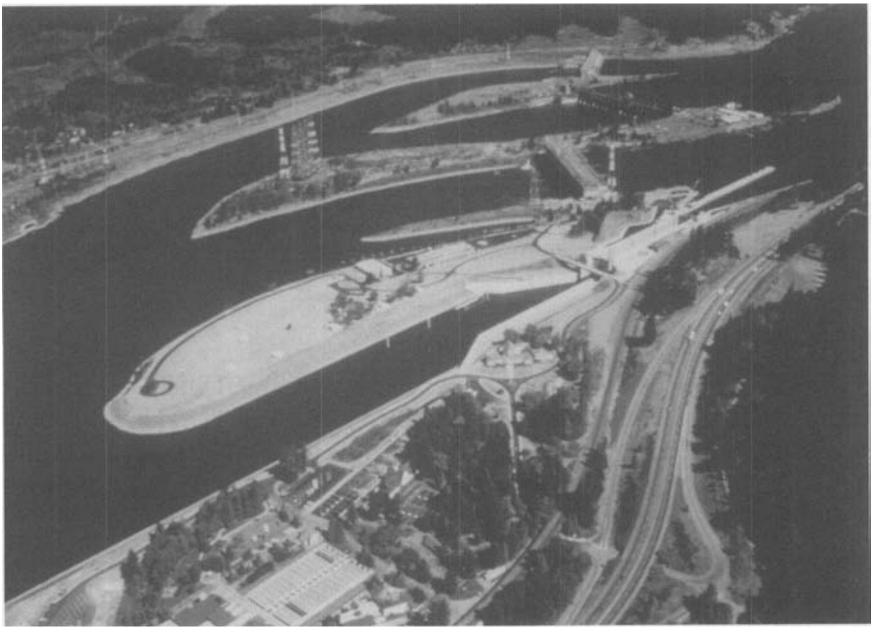


FIGURE 20-5. Bonneville Complex: Lock and Powerhouse in Foreground, Spillway Center, Second Powerhouse in Distance.

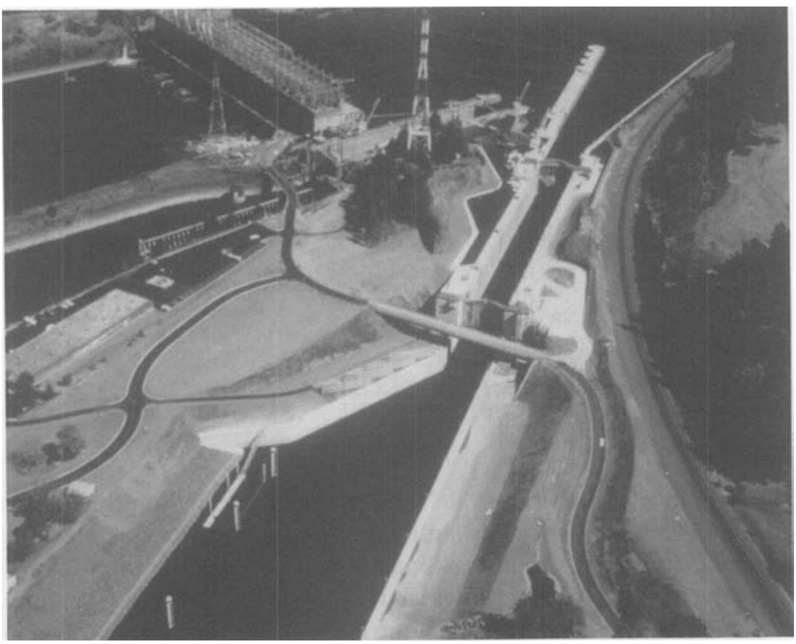


FIGURE 20-6. New Bonneville Lock.

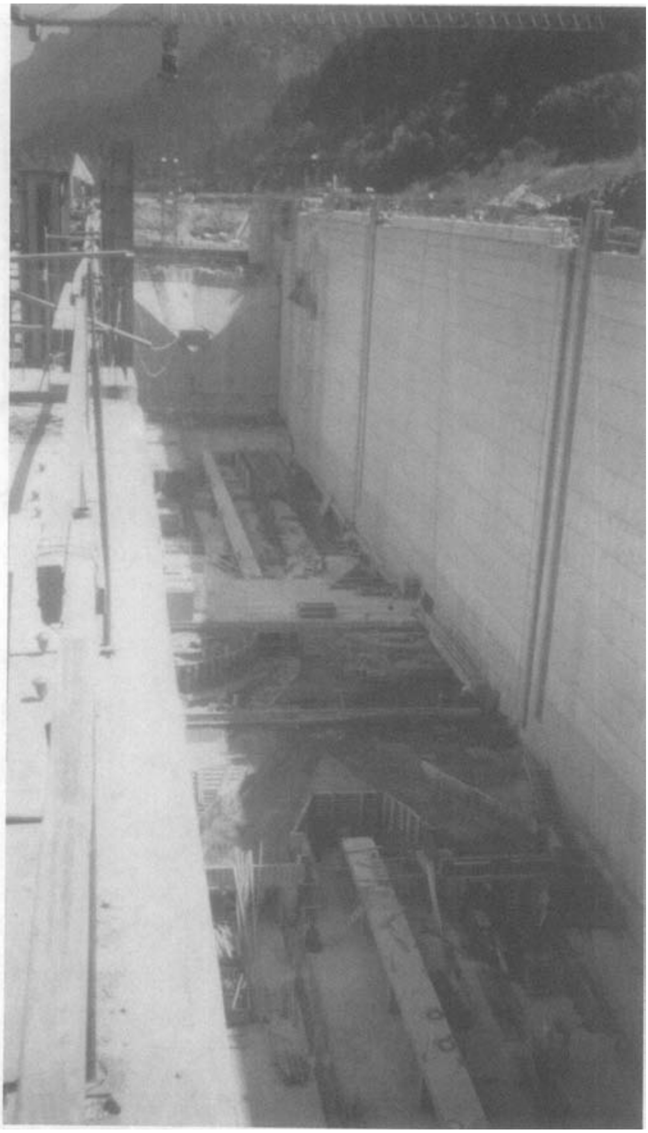


FIGURE 20-7. Bonneville Lock, Lock Chamber Under Construction.

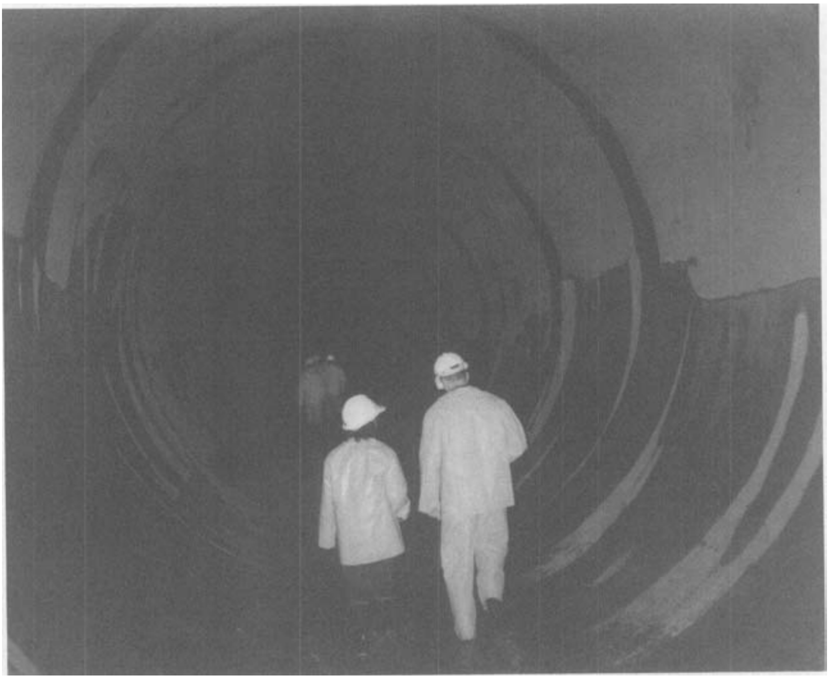


FIGURE 20-8. Bonneville Lock, Fill-and-Empty Conduits Mined Through Rock (15 ft dia).

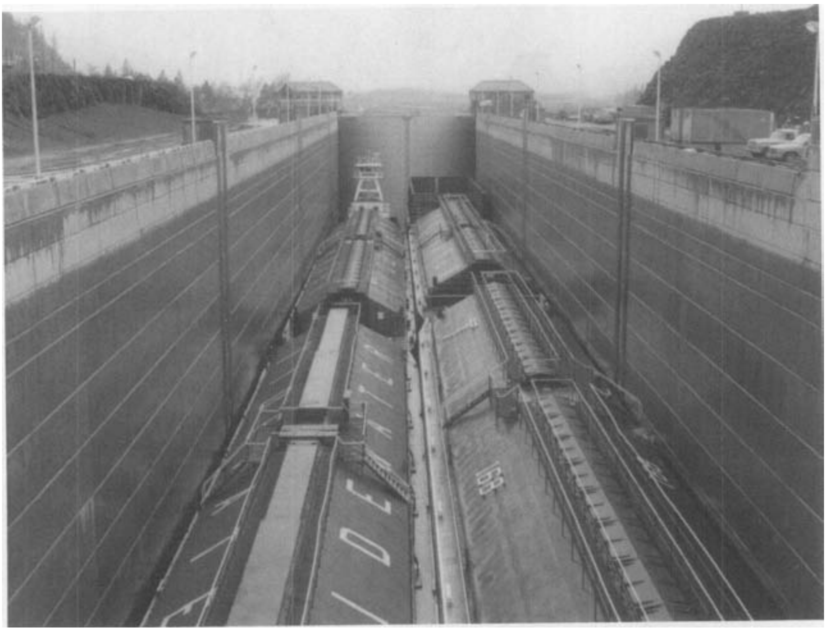


Figure 20-9. Bonneville Lock, Design Tow in Lock Chamber.

Pertinent Data

Lock Usable Length	675 ft
Lock Width	86 ft
Lock Maximum Lift	70 ft
Design Tow Dimensions	84 ft wide × 650 ft long
Design Tow Draft	14 ft

Lock Filling System

The filling system is a bottom longitudinal type with four discharge conduits in the chamber (tuning fork configuration). The filling conduits between the filling valves and discharge conduits are mined through rock on each side of the chamber.

Construction Chronology

	Start	Complete
Relocate ½ mile of railroad track	July 86	Aug 87
Initial lock excavation—1 million cu of rock	July 87	
Upstream guardwall and prehistoric landslide stabilization structure	April 88	
Lock and approaches	Sept 88	March 93
Open to traffic		March 93

Project Cost

$330 million including designs, construction management, relocation, channel work, landslide stabilization, and lock construction cost.

Models Used for Design

- General navigation, 1 to 80 scale.
- Lock fill and empty, 1 to 25 scale.

Unique Features

- Upstream approach flows up to 6 fps. Powerhouse flows create high velocities in the confined channel shared with the lock.
- A series of upstream submerged groins was used to reduce velocities in the lock approach. Groin dimensions and location were determined in the general navigation model.

FIGURE 20-10. Bonneville Lock, 400 ft Long Floating Section of Upstream Guide Wall.

- Upstream guard wall also was used to stabilize a prehistoric landslide zone. Tendons were used to tie wall into good rock in hillside.
- Upstream guide wall was made up of floating pontoons connected to sheetpile cells. One 400 ft long pontoon was not fixed to cells but allowed to float in slots in cells. The floating pontoon was needed to reduce currents under the pontoon that would pin the tow to the wall at low pool levels. This tow pinning problem was discovered in model testing.
- Lock was located in a large rock formation. This allowed the walls attached to rock with rock bolts and fill-and-empty conduits mined through rock instead of being located in a gravity wall section.

CASE HISTORY 3. MELVIN PRICE LOCK AND DAM

Description

The Melvin Price Locks and Dam replaces the old locks and dam no. 26 on the Mississippi River near St. Louis, MO. The Melvin Price project is about 2 miles downstream from the old No. 26 lock. The old lock and dam No. 26 with its 2 lock chambers of 600 ft and 360 ft in length had become a bottleneck to Mississippi River traffic. In 1983, when traffic reached over 73 million tons annually, the average delay for barge traffic was 15 hours due to inadequate locking capacity. The new Melvin Price Locks with two chambers (one 1,200 ft long and one 600 ft long) were constructed to relieve the traffic delays. Figure 20-11 is a photograph of this project.

Pertinent Data

	Main Lock (ft)	Auxiliary Lock (ft)
Lock Usable Length	1,200	600
Lock Width	110	110
Design Tow Dimensions	1,200 × 105	600 × 105
Design Tow Draft	9	9
Lock Lift	24	24
Lock Fill System	side port	side port

Construction Chronology

Total construction of the Melvin Price Locks and dam took about 14 years, but the main lock became operational after 10 years. The old lock No. 26 continued to operate between 1980, when construction on the new dam began, until 1990, when the main lock of the new structure was opened. In the Spring of 1990, after more than 50 years in operation, old locks and dam No. 26 were removed.

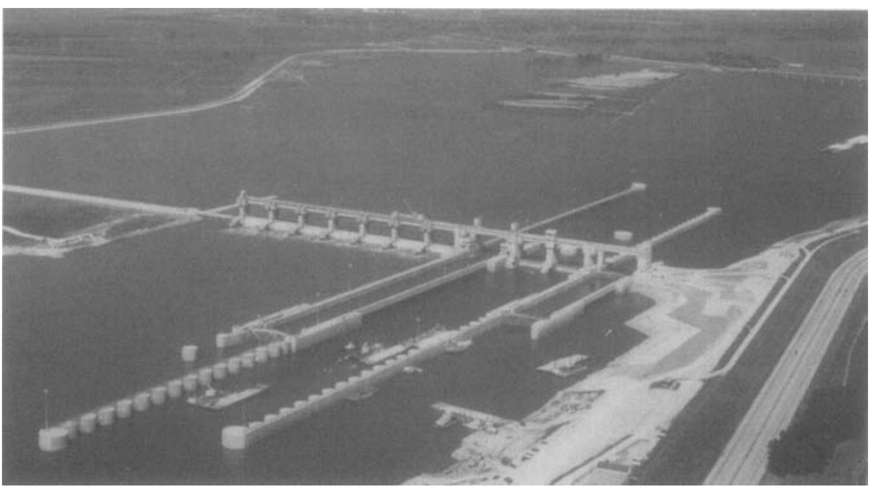


FIGURE 20-11. Melvin Price Locks and Dam.

The Melvin Price Locks and dam were built in three stages as follows.

Stage	Start	Complete
6½ gate bays of the dam	1980	1985
Main lock and 2½ gate bays of the dam	1985	1990
Auxiliary lock and 1½ gate bays of the dam	1990	1994

Project Cost

Federal project cost for this project is $956 million as of 1996. Non-Federal cost was $5.7 million. These project costs included planning, engineering, design, construction management, real estate, relocations, recreation facilities, and structure construction costs. The cost allocated to the dam structure was $290.8 million. The cost allocated to the locks was $413.9 million.

Model Studies Used For Design

Spillway Model Study. Tests were conducted on two physical models (1:36 scale) to determine discharge characteristics of the spillway, stilling basin performance, riprap requirements downstream from the structure, and gate vibration tendencies.

Navigation Model. The model reproduced about 6.7 miles of the Mississippi River to an undistorted scale of 1:120. The model had a movable bed in the vicinity of the project and a fixed bed upstream and downstream. The model study was concerned with the composition and configuration of the dam, arrangement and separation of the locks and lock walls, and navigation conditions in the lock approaches. Also studied was a proposed hydropower project, navigation and potential scour concerns during the third phase construction sequence, and navigation during and after removal of the old locks and dam No. 26. The model was also used to develop the proper length guide wall and port opening.

Cofferdam Floodgate Model Study. The investigation was conducted to determine the stability of the stone protection and potential for leaching of fill material through the stone cover for the design flow conditions.

Mathematical Models. TABS II was used to analyze sedimentation movement during the three stage construction sequence.

Unique Features

Separated Locks. Discussions with tow pilots indicated that simultaneous approaches and departures could take place if the locks were separated rather than being side by side. Two gates of the dam are in the separation to provide better flow conditions for navigation.

Lock Automation. The lock control system uses modern electronic equipment that enables the lock operator to lock tows through the lock in modes ranging from semi-automatic to manual. In the semi-automatic mode, a number of operations are programmed to occur once the lockage button is activated. In the manual mode, a separate action is required for each lock operation, that is, the opening of a gate or the closing of a valve.

Guidewalls Construction Without A Cofferdam. A plan to construct the guidewalls in the wet without a cofferdam resulted from a value engineering study. The guidewall consists of precast concrete beams placed on concrete-filled sheetpile cells. Slurry concrete was used to fill the cells, with a notch constructed to support the concrete beams.

CASE HISTORY 4. RED RIVER WATERWAY

Description

The Red River Waterway provides a 9 ft deep by 200 ft wide navigation channel extending 236 miles from the Mississippi through Old River and Red River to near Shreveport, LA. The river passes through lands containing alluvial soil rich in iron oxide, giving the water its rusty color and the river its name. The waterway has 5 locks and extensive channel realignment, bank stabilization, and river training works. Figures 20-12 through 20-14 are photographs of this project.

FIGURE 20-12. Lock and Dam No. 3 on Red River.

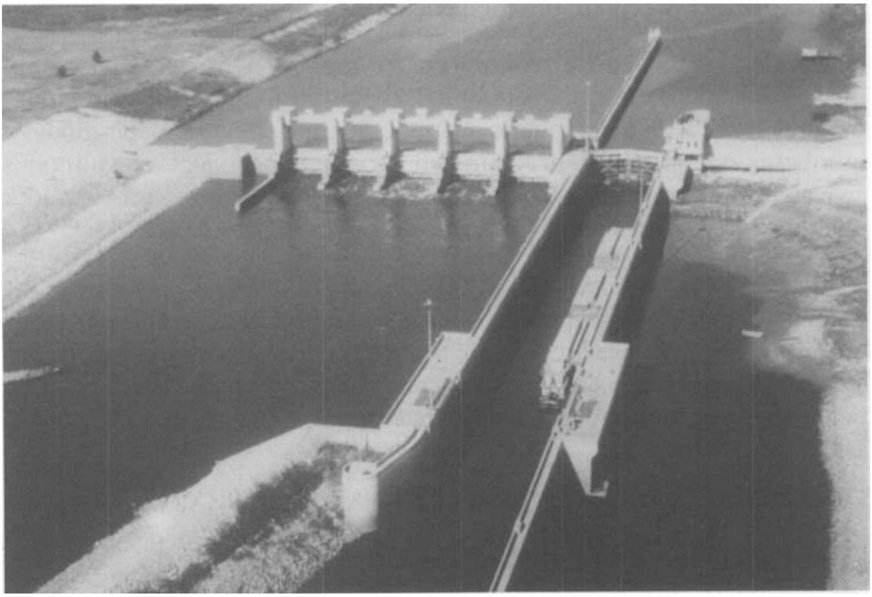


FIGURE 20-13. Lock and Dam No. 4 on Red River.

FIGURE 20-14. Tow on Red River.

Pertinent Data

Locks Usable Length	685 ft
Lock Width	84 ft
Design Tow Draft	9 ft
Lock Filling System	side port

Lock Lifts

Lock	Lift (ft)
Lindy C. Boggs	36
John Overton	24
Lock and Dam No.3	31
Lock and Dam No. 4	25
Joe D. Waggoner, Jr.	25

Construction Chronology

	Start	Open to Navigation
Lindy C. Boggs	June 1977	Dec 1987
John Overton	Nov. 1982	Nov. 1987
Lock and Dam No.3	Sept. 1985	Dec. 1994
Lock and Dam No. 4	June 1990	Dec. 1994
Joe D. Waggoner, Jr	July 1990	Dec. 1994

Project Cost

Project cost estimated through FY 1996	$1,989 Million
Estimated cost for 5 locks and dams	$543 Million

Other costs include relocation, bridges, real estate, channel work, design, and construction management.

Models Used For Design

The US Army Corps of Engineers Waterway Experiment Station at Vicksburg, MS, assisted in the locks and dam design by building scale models of portions of the waterway. Mathematical models also aided the design effort. The following briefly describes the use of each model.

Spillway Model. This fixed bed model was used to refine the design of the locks' and dams' structural features including the spillway, stilling basin, and downstream riprap protection.

Navigation Model. This fixed bed model was used to refine the design of lock approaches and the channel near the lock and dam to provide safe and efficient navigation conditions.

Movable Bed Model. This model used fine ground coal for the channel bed to simulate the movement of sediment. The model was used to optimize design of the lock approaches and the channel near the lock and dam to reduce the potential for sediment deposition.

Mathematical Models. These models were used to study sediment movements and current directions and velocities.

Other Models. A typical river reach model was used to develop design criteria concerning river training works such as dikes and revetments. A fixed bed navigation model was used to determine the proper configuration for several bridges.

Unique Features

- Sediment control was one of the unique problems faced in the design of this waterway. The Red River had historically had a large fine-grained sediment load. Locks and dams 2,3,4, and 5 have scour jet systems.
- Locks and dams 4 and 5 have hinged crest gates to increase dissolved oxygen content during low flow periods.
- The downstream guide wall at the Lindy C. Boggs lock is a floating structure to accommodate the large tailwater fluctuation caused by the Mississippi backwater.
- The realigned navigation channel involved severing 36 bendways along the river. A closure dam at the upstream end of the old bendway preserves the oxbows and the downstream end of the oxbows remain open to the river. Preservation of the bendway improves fisheries and recreational opportunities.

CASE HISTORY 5. LELAND BOWMAN LOCK

Description

The Leland Bowman lock is located on the Louisiana section of the Gulf Intercoastal Waterway south of Abbeville, Vermilion Parish, LA. This lock replaced the Old Vermilion Lock. Figure 20-15 has photographs of this project.

FIGURE 20-15. *Leland Bowman Lock: (top) Lock Under Construction; (bottom) Completed Lock.*

Pertinent Data

Lock Usable Length	1,198 ft
Lock Width	110 ft
Lock Maximum Lift	6 ft
Design Tow Dimensions	54 ft wide × 980 ft long
Design Tow Draft	9 ft

Lock Filling System

The end filling system uses two sets of sector gates to fill and empty. The sector gates weigh 105 tons each.

Construction Chronology

	Start	Complete
Notice to proceed	30 Oct. 1981	
Excavation	20 Mar. 1982	15 Mar. 1983
Pile driving	10 Aug. 1983	30 Dec. 1983
Timber forming	26 Aug. 1983	6 Jan. 1984
Erected Gate #1	7 Dec. 1983	
Erected Gate #2	17 Dec. 1983	
Sheet pile dolphin	8 Sept. 1984	11 Jan. 1985
Walkways and rails	6 Jan. 1984	6 Jun. 1984
Riprap	20 Aug. 1984	24 Nov. 1984
Remove land plug	13 Jan. 1985	
Lock open to traffic	12 Feb. 1985	

Project Cost

The bid on the lock replacement was $26.2 million. The construction cost with modifications was $30.0 million.

Model Used for Design

No model studies were made.

Unique Features

The lock controls saltwater that would flow into the Mermentau River Basin and the lock also drains excess water that accumulates in the basin.

CASE HISTORY 6. WILLAMETTE FALLS LOCKS

Description

The Willamette Falls Locks are located on the Willamette River in Oregon City, OR. Figures 20-16 through 20-18 are photographs of this project.

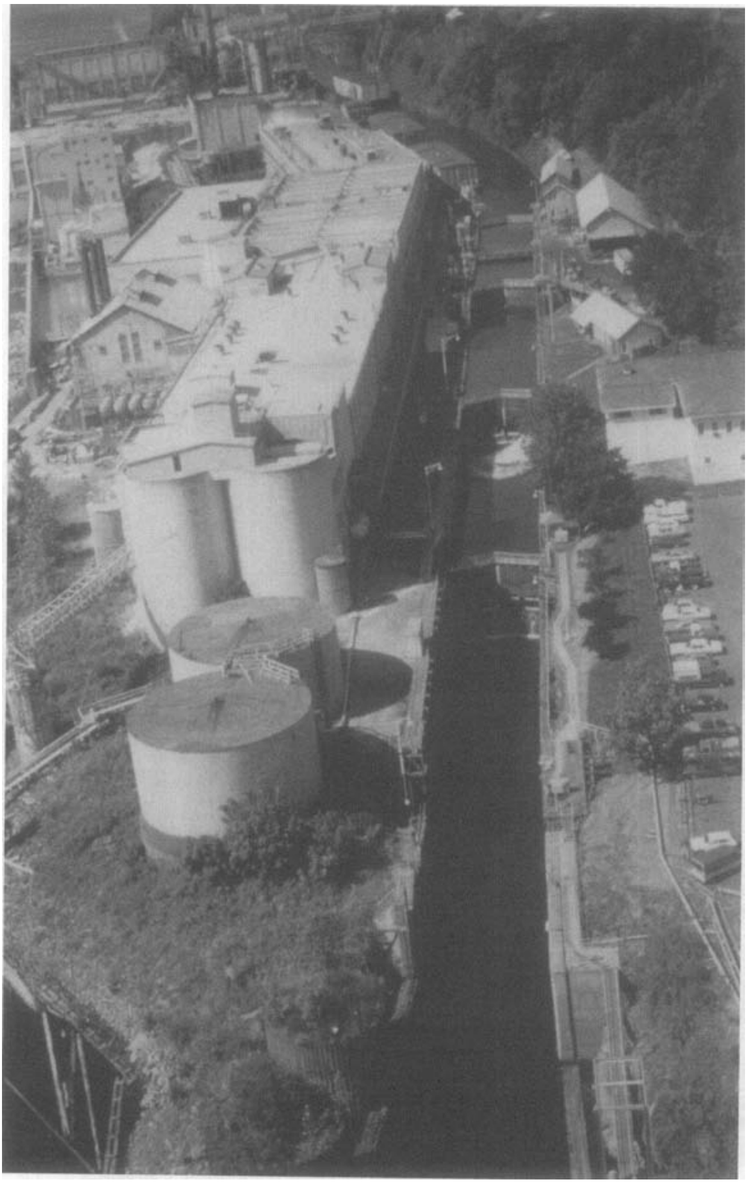


FIGURE 20-16. Willamette Falls Locks, Aerial View.

Pertinent Data

Lock Usable Length	210 ft
Lock Usable Width	40 ft
Maximum Lift	41 ft for four chambers, about 10 ft lift for each chamber
Design Tow Dimensions	37 ft wide × 175 ft long
Design Tow Draft	6 ft

Lock Fill System

End filling system using multiple valves near the bottom of the miter gate.

Construction Chronology

- Project built by Peoples Transportation Company in early 1870s
- Opened to traffic January 1, 1873.

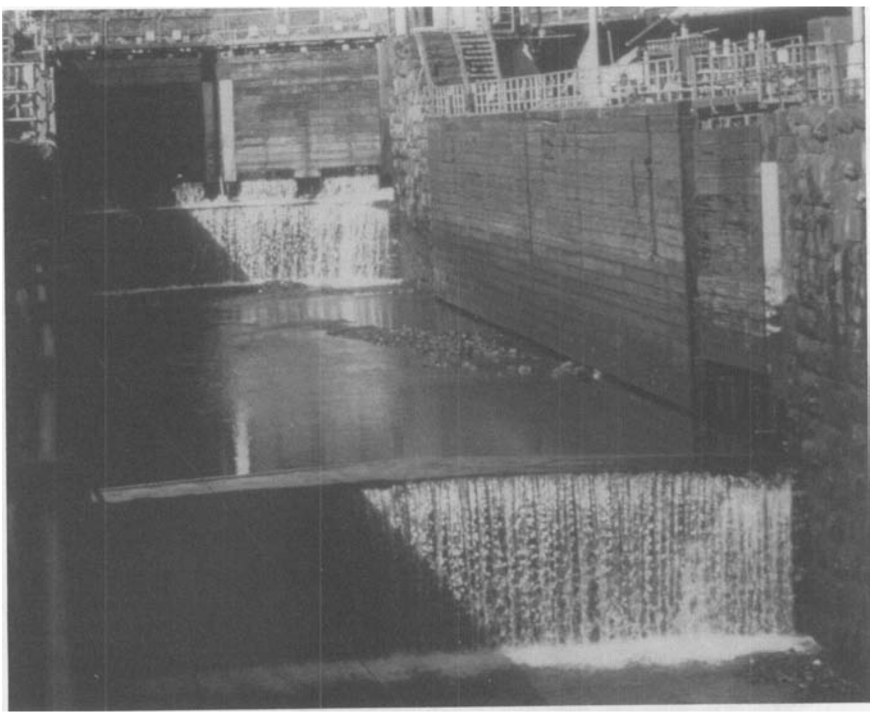


FIGURE 20-17. Willamette Falls Locks, Lock Chamber.

FIGURE 20-18. Willamette Falls Locks, Log Raft.

- Operated by a number of owners until purchased by the US Army Corps of Engineers in 1915.
- Corps renovation of the lock was completed in 1921.

Project Cost

The Corps purchased the lock for $375,000 in 1915.

Unique Features

- Four lock chambers in tandem, 10 ft lift per chamber.
- From 1940 to 1970 the lock passed an average of 1.5 million tons of commerce per year, mostly rafted logs.
- In 1974 the project was placed on the National Register of Historic Places.

This page intentionally left blank

REFERENCES

ASCE (1993) *Report on Ship Channel Design.* Manuals and Reports on Engineering Practice No. 80, American Society of Civil Engineers.

ASCE (1995) *Guidelines for Design of Intakes for Hydroelectric Plants.* Committee on Hydropower Intakes of the Energy Division, American Society of Civil Engineers.

Cox, R.G. (1972) *Pertinent Data on Spillway Tainter Gates for Corps of Engineers Projects.* MPH-72-7, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Davis, J.P. (1989) *Hydraulic Design of Navigation Locks.* MP, HL-89-5, US Army Waterways Experiment Station, Vicksburg, MS.

Hydraulic Design Criteria (HDC) sheets and charts. Available from US Army Engineer Waterways Experiment Station, Vicksburg, MS.

McCartney, B.L. (1986) "Inland waterway navigation project design." *ASCE J Waterways Port Coastal Ocean Eng.* 112(6) Nov.

Office, Chief of Engineers, US Army. (1949) *Hydraulic Design of Surges in Canals.* EM 1110-2-1606, Washington, DC, Mar.

Office, Chief of Engineers, US Army. (1962) *Vertical Lift Crest Gates.* EM 1110-2-1701, Washington, DC, 7 Dec.

Office, Chief of Engineers, US Army. (1975) *Hydraulic Design of Lock Culvert Values.* EM 1110-2-1605, Washington, DC, 15 Aug.

Office, Chief of Engineers, US Army. (1980) *Hydraulic Design of Reservoir Outlet Works.* EM 1110-2-1602, Washington, DC, 15 Oct.

Office, Chief of Engineers, US Army. (1980) *Layout and Design of Shallow-Draft Waterways.* EM 1110-2-1611, Washington, DC, 31 Dec.

Office, Chief of Engineers, US Army. (1982) *Ice Engineering,* EM 1110-1-1612, Washington, DC, 15 Oct.

Office, Chief of Engineers, US Army. (1983) *Dredging and Dredged Material Disposal.* EM 1110-2-5025, Washington, DC, 25 Mar.

Office, Chief of Engineers, US Army. (1984) *Lock Gates and Operating Equipment* EM 1110-2-1703, Washington, DC, 29 Feb.

Office, Chief of Engineers, US Army. (1987) *Hydraulic Design of Navigation Dams.* EM 1110-2-1605, Washington, DC, 12 May.

Office, Chief of Engineers, US Army. (1987) *Environmental Engineering for Deep-Draft Navigation Projects.* EM 1110-2-1202, Washington, DC, 29 May.

Office, Chief of Engineers, US Army. (1990) *Hydraulic Design of Spillways*. EM 1110-2-1603, Washington, DC, 16 Jun.

Office, Chief of Engineers, US Army. (1995) *Hydraulic Design of Navigation Locks*. EM 1110-2-1604, Washington, DC, 30 Jun.

Office, Chief of Engineers, US Army. (1995) *Evaluation and Repair of Concrete Structures*. EM 1110-1-2002, Washington, DC, 30 June.

Office, Chief of Engineers, US Army. (1995) *Planning and Design of Navigation Lock Walls and Appurtenances*. EM 1110-2-2602, Washington, DC, 30 Dec.

Rivers and Harbors Appropriation Act, Section 5. (1915) approval 4 March (PL 291, 1915).

Appendix A

INVENTORY OF CORPS OF ENGINEERS LOCKS

Acronyms for filling system types follows; a preceding * represents recent design type.

G = Butterfly valve(s) in gate
VG = Other valve(s) in gate (slide valves normally)
BW = Butterfly valve(s) in wall
CW = Cylinder valve(s) in wall
LC = Loop culvert(s)
LCSG = Loop culvert(s) and sector gate
*SG = Sector gates
*SP = Side ports
*SPF = Side ports with flume
MP = Multiport system
*BL1 = Centered lateral-manifolds; one-culvert
*BL2 = Centered lateral-manifolds; two-culvert
BLC = Centered lateral-manifolds; high-lift modified
SBLC = Split lateral-manifolds
OC = Longitudinal centered and ported culvert
VB4 = Vertical flow dividers; four longitudinal manifolds
VB8 = Vertical flow dividers; eight longitudinal manifolds
*HB4 = Horizontal flow dividers; four longitudinal manifolds
*HB8 = Horizontal flow dividers; eight longitudinal manifolds

Project	Community in Vicinity	Year Opened	Width of Chamber (ft)	Locks Usable Length (ft)	Lift at Normal Pool (ft)	Filling System Acronym
Alabama-Coosa Rivers, AL						
Claiborne Lock and Dam	Claiborne, AL	1973	84	600	30	VB4
Millers Ferry Lock and Dam	Camden, AL	1969	84	600	45	VB4
Jones Bluff Lock and Dam	Benton, AL	1974	84	600	45	VB4
Allegheny River, PA and NY						
Lock and Dam No. 2	Aspinwill, PA	1934	56	360	11	SP
Lock and Dam No. 3	Cheswick, PA	1934	56	360	13	SP
Lock and Dam No. 4	Natrona, PA	1927	56	360	10	SP
Lock and Dam No. 5	Freeport, PA	1927	56	360	12	SP
Lock and Dam No. 6	Clinton, PA	1928	56	360	12	SP
Lock and Dam No. 7	Kitanning, PA	1931	56	360	13	SP
Lock and Dam No. 8	Templeton, PA	1937	56	360	18	SP
Lock and Dam No. 9	Rimer, PA	1938	56	360	22	SP
Apalachicola, Chattahoochee, and Flint Rivers, GA, and FL						
Jim Woodruff Lock and Dam	Chattahoochee, FL	1957	82	450	33	SP
George W. Andrews Lock and Dam	Columbia, GA	1963	82	450	25	SP
Walter F. George Lock and Dam	Fort Gaines, GA	1963	82	450	88	SBLC
Atlantic Intracoastal Waterway						
Albemarle and Chesapeake Canal Rt: Great Br. Lock	Great Bridge, VA	1932	75	600	3	LC
Dismal Swamp Canal Route: Deep Creek Lock	Deep Creek, VA	1940	52	300	12	VG
South Mills Lock	South Mills, NC	1941	52	300	12	VG
Bayou Teche, LA						
Berwick Lock	Berwick, LA	1951	45	300	7	SG
Keystone Lock	New Iberia, LA	1993	36	160	8	SP
Black Rock Channel and Tonawanda Harbor, NY						
Black Rock Lock	Buffalo, NY	1914	68	625	5	—
Black Warrior, Warrior, and Tombigbee Rivers, AL						
Coffeeville (Jackson) Lock and Dam	Coffeeville, AL	1965	110	600	34	SP
Demopolis Lock and Dam	Demopolis, AL	1962	110	600	40	BL2

Project	Community in Vicinity	Year Opened	Width of Chamber (ft)	Locks Usable Length (ft)	Lift at Normal Pool (ft)	Filling System Acronym
Armistead I Seldon Lock and Dam	Eutaw, AL	1962	110	600	22	SP
Wm. Bacon Oliver Lock and Dam	Tuscaloosa, AL	1991	110	600	28	SP
Holt Lock and Dam	Holt, AL	1969	110	600	68	BLC
John Hollis Bankhead Lock and Dam	Adger, AL	1975	110	600	68	VB4
Gainesville Lock and Dam	Gainesville, AL	1978	110	600	36	VB4
Aliceville Lock and Dam	Aliceville, AL	1979	110	600	27	SP
Columbus Lock and Dam	Columbus, MS	1980	110	600	27	SP
Aberdeen Lock and Dam	Aberdeen MS	1985	110	600	27	SP
Lock and Dam A	Amory, MS	1985	110	600	30	SP
Lock and Dam B	Amory, MS	1985	110	600	25	SP
Lock and Dam C	Fulton, MS	1985	110	600	25	SP
Lock and Dam D	Fulton, MS	1985	110	600	30	SP
Lock and Dam E	Fulton, MS	1985	110	600	30	SP
Bay Springs Lock and Dam	Fulton, MS	1985	110	600	84	HB4
Canaveral Harbor, FL						
Canaveral Lock	Cocoa, FL	1965	90	600	3	—
Cape Fear River, NC						
Lock and Dam No. 1	Kings Bluff, NC	1934	40	200	11	BG
Lock and Dam No. 2	Browns Landing, NC	1917	40	200	9	BG
William O. Huske Lock and Dam	Tolars Landing, NC	1935	40	300	9	BG
Central and Southern Florida						
S-61 Lock	St. Cloud, FL	1963	30	90	2	SG
S-65 Lock	Frostproof, FL	1964	30	90	6	SG
S-65A Lock	Avon Park, FL	1967	30	90	6	SG
S-65B Lock	Sebring, FL	1965	30	90	6	SG
S-65C Lock	Sebring, FL	1965	30	90	7	SG
S-65D Lock	Okeechobee, FL	1964	30	90	6	SG
S-65E Lock	Okeechobee, FL	1964	30	90	5	SG
Columbia River, OR and WA						
Bonneville Lock and Dam	Bonneville, OR					
Old Lock	Bonneville, OR	1938	76	500	69	OC
New Lock	Bonneville, OR	1993	86	675	69	VB4

Project	Community in Vicinity	Year Opened	Width of Chamber (ft)	Locks Usable Length (ft)	Lift at Normal Pool (ft)	Filling System Acronym
The Dalles Lock and Dam	The Dalles, OR	1957	86	675	88	SBLC
John Day Lock and Dam	Rufus, OR	1968	86	669	110	BLC
McNary Lock and Dam	Umatilla, OR	1953	86	683	75	BLC
Cross Florida Barge Canal						
Inglis Lock, Dam, and Spillway	Inglis, FL	1968	84	600	28	SP
Eureka Lock and Dam		1971	84	600	20	SP
Henry H. Buckman Lock	Palatka, FL	1972	84	600	20	SP
Cumberland River, KY and TN						
Lock and Dam	Kuttawa, KY	1964	110	800	57	SBLC
Cheatham Lock and Dam	Ashland City, TN	1959	110	800	26	SP
Old Hickory Lock and Dam	Old Hickory, TN	1957	84	400	60	MP
Cordell Hull Lock and Dam	Carthage, TN	1973	84	400	59	MP
Fox River, WI						
DePere Lock	DePere, WI	1936	36	146	9	MP
Little Kaukauna Lock	De Pere, WI	1936	36	146	7	BG
Rapid Croche Lock	Wrightstown, WI	1934	36	146	8	BG
Kaukauna Fifth Lock	Kaukauna, WI	1898	36	144	9	BG
Kaukauna Fourth Lock	Kaukauna, WI	1879	37	144	10	BG
Kaukauna Third Lock	Kaukauna, WI	1879	37	144	10	BG
Kaukauna Second Lock	Kaukauna, WI	1903	35	144	11	BG
Kaukauna First Lock	Kaukauna, WI	1883	35	144	11	BG
Kaukauna Guard Lock	Kaukauna, WI	1891	40	—	—	—
Little Chute Combined Lock						
Lower	Little Chute, WI	1879	35	147	11	BG
Upper	Little Chute, WI	1879	36	144	11	BG
Little Chute, Second Lock	Little Chute, WI	1881	35	144	14	BG
Little Chute, First (Guard) Lock	Little Chute, WI	1904	35	—	7	BG
Cedars Lock	Little Chute, WI	1888	35	144	10	BG
Appleton Fourth Lock	Appleton, WI	1907	35	144	8	BG
Appleton Third Lock	Appleton, WI	1900	35	144	9	BG
Appleton Second Lock	Appleton, WI	1901	35	145	10	BG
Appleton First Lock	Appleton, WI	1884	35	145	10	BG
Menasha Lock	Menasha, WI	1899	35	144	8	BG

Project	Community in Vicinity	Year Opened	Width of Chamber (ft)	Locks Usable Length (ft)	Lift at Normal Pool (ft)	Filling System Acronym
Freshwater Bayou Lock, LA	Intracoastal City, LA	1968	84	600	—	SG
Green and Barren Rivers, KY						
Green River						
Lock and Dam No. 1	Spottsville, KY	1956	84	600	12	SP
Lock and Dam No. 2	Calhoun, KY	1956	84	600	14	SP
Lock and Dam No. 3	Rochester, KY	1836	36	138	17	VG
Lock and Dam No. 4	Woodbury, KY	1839	35	138	16	VG
Barren River						
Lock and Dam No. 1	Greencastle, KY	1934	56	360	15	—
Gulf Intracoastal Waterway						
Inner Harbor Navigation Channel Lock	New Orleans, LA	1923	75	640	9	—
Harvey Lock	Harvey, LA	1935	75	425	10	SP
Algiers Lock	Algiers, LA	1956	75	800	10	SG
Bayou Boeuf Lock	Morgan City, LA	1956	75	1,156	6	SG
Bayou Sorrel Lock	Plaquemine, LA	1952	56	797	10	SG
Port Allen Lock	Port Allen, LA	1961	84	1,202	5	SG
Leland Bowman	Abbeville, LA	1984	110	1,198	3	SG
Calcasieu Lock	Lake Charles, LA	1950	75	1,206	6	SG
Colorado River, TX						
East Lock	Matagorda, TX	1954	75	1,200	5	—
West Lock	Matagorda, TX	1954	75	1,200	5	—
Hudson River, NY						
Troy Lock and Dam	Troy, NY	1917	44	493	17	SP
Illinois Waterway, IL						
LaGrange Lock and Dam	Beardstown, IL	1939	110	600	10	SP
Peoria Lock and Dam	Peoria, IL	1939	110	600	11	SP
Starved Rock Lock and Dam	Utica, IL	1933	110	600	19	SP
Marseilles Lock	Marseilles, IL	1933	110	600	24	SP
Dresden Island Lock and Dam	Morris, IL	1933	110	600	22	SP
Brandon Road Lock and Dam	Joliet, IL	1933	110	600	34	SP
Lockport Lock	Lockport, IL	1933	110	600	40	SP
Thomas J. O'Brien Lock and Dam	Chicago, IL	1960	110	1,000	2	LCSG

Project	Community in Vicinity	Year Opened	Width of Chamber (ft)	Locks Usable Length (ft)	Lift at Normal Pool (ft)	Filling System Acronym
Inland Route, MI						
Crooked River Lock and Weir	Alanson, MI	1967	17.8	66	66	—
Kanawha River, WV						
Winfield Lock and Dam	Winfield, WV	1937	56	360	28	SP
Marmet Lock and Dam	Marmet, WV	1934	56	360	24	SP
London Lock and Dam	London, WV	1934	56	360	24	SP
Kentucy River, KY						
Lock and Dam No. 1	Carrolton, KY	1839	38	145	8	BG
Lock and Dam No.2	Lockport, KY	1839	38	145	14	BG
Lock and Dam No. 3	Gest, KY	1844	38	145	13	BG
Lock and Dam No. 4	Frankfort, KY	1844	38	145	13	BG
Lock and Dam No. 5	Tyrone, KY	1844	38	145	15	BG
Lock and Dam No. 6	High Bridge, KY	1891	52	147	14	CW
Lock and Dam No. 7	High Bridge, KY	1897	52	147	15	BW
Lock and Dam No. 8	Camp Nelson, KY	1900	52	146	19	BW
Lock and Dam No. 9	Valley View, KY	1907	52	148	17	CW
Lock and Dam No. 10	Ford, KY	1907	52	148	17	CW
Lock and Dam No. 11	Irvine, KY	1906	52	148	18	CW
Lock and Dam No. 12	Ravenna, KY	1910	52	148	17	CW
Lock and Dam No. 13	Willow, KY	1915	52	148	18	CW
Lock and Dam No. 14	Heidelberg, KY	1917	52	148	17	CW
Lake Washington Ship Canal						
Hiram M. Chettenden Locks						
Large Lock	Seattle, WA	1916	80	760	26	SP
Small Lock	Seattle, WA	—	28	123	26	SP
McClellan-Kerr Arkansas River, AR and LA						
Norrell Lock and Dam	Arkansas Post, AR	1967	110	600	30	SP
Lock No. 2	Arkansas Post, AR	1967	110	600	20	SP
Lock and Dam No. 3	Grady, AR	1968	110	600	20	SP
Lock and Dam No. 4	Pine Bluff, AR	1968	110	600	14	SP
Lock and Dam No. 5	Redfield, AR	1968	110	600	17	SP
David D. Terry Lock and Dam	Little Rock, AR	1968	110	600	18	SP
Murray Lock and Dam	Little Rock, AR	1969	110	600	16	SP
Toad Suck Ferry Lock and Dam	Conway, AR	1969	110	600	19	SP
Lock and Dam No. 9	Morrilton, AR	1969	110	600	19	SP
Lock and Dam No. 13	Fort Smith, AR	1969	110	600	19	SP
Dardanelle Lock and Dam	Russellville, AR	1969	110	600	54	VB4

Project	Community in Vicinity	Year Opened	Width of Chamber (ft)	Locks Usable Length (ft)	Lift at Normal Pool (ft)	Filling System Acronym
Ozark Lock and Dam	Ozark, AR	1975	110	600	34	SP
W.D. Mayo Lock and Dam	Fort Smith, AR	1970	110	600	10	SP
Chouteau Lock and Dam	Muskogee, OK	1970	110	600	21	SP
Newt Graham Lock and Dam	Inola, OK	1970	110	600	21	SP
Robert S. Kerr Lock and Dam and Reservoir	Sallisaw, OK	1970	110	600	48	BLC
Webbers Falls Lock and Dam	Webbers Falls, OK	1970	110	600	30	SP
Mississippi River Between Ohio and Missouri Rivers						
Lock and Dam No. 27	Granite City, IL	1963	110	1,200	21	SP
		1963	110	600	21	—
Melvin Price Lock and Dam	Alton, IL	1990	110	1,200	24	SP
		1990	110	600	24	SP
Lock and Dam No. 25	Cap Au Gris, MO	1939	110	600	15	SP
Lock and Dam No. 24	Clarksville, MO	1940	110	600	15	SP
Lock and Dam No. 22	Saverton, MO	1938	110	600	10	SP
Lock and Dam No. 21	Quincy, IL	1938	110	600	10	SP
Lock and Dam No. 19	Keokuk, IA	1913	110	358	38	—
		1957	110	1,200	38	—
Lock and Dam No. 18	Burlington, IA	1937	110	600	10	SP
Lock and Dam No. 17	New Boston, IL	1939	100	600	9	SP
Lock and Dam No. 16	Muscatine, IA	1937	110	600	9	SP
Lock and Dam No. 15	Rock Island, IL	1934	110	600	16	SP
		1934	110	360	16	SP
Lock and Dam No. 14	LeClaire, IA	1922	80	320	11	SP
	LeClaire, IA	1939	110	600	11	SP
Lock and Dam No. 13	Clinton, IA	1939	110	600	11	SP
Lock and Dam No. 12	Bellevue, IA	1938	110	600	9	SP
Lock and Dam No. 11	Dubuque, IA	1937	110	600	11	SP
Lock and Dam No. 10	Guttenberg, IA	1936	110	600	8	SP
Lock and Dam No. 9	Lynxville, WI	1938	110	600	9	SP
Lock and Dam No. 8	Genos, WI	1937	110	600	11	SP
Lock and Dam No. 7	Dresbach, MN	1937	110	600	8	SP
Lock and Dam No. 6	Trempealeau, WI	1936	110	600	6	SP
Lock and Dam No. 5A	Winona, MN	1936	110	600	5	SP
Lock and Dam No. 5	Minneiska, MN	1935	110	600	9	SP
Lock and Dam No. 4	Alma, WI	1935	110	600	7	SP
Lock and Dam No. 3	Red Wing, MN	1938	110	600	8	SP
Lock and Dam No. 2	Hastings, MN	1930	110	600	12	SP

Project	Community in Vicinity	Year Opened	Width of Chamber (ft)	Locks Usable Length (ft)	Lift at Normal Pool (ft)	Filling System Acronym
Lock and Dam No. 1	Minneapolis–St. Paul	1948	56	400	36	SP
		1932	56	400	36	SP
St. Anthony Falls Lower Lock and Dam	Minneapolis, MN	1917	56	400	27	BL2
St. Anthony Falls Upper Lock and Dam	Minneapolis, MN	1963	56	400	49	BLC
Monongehela River, PA and WV						
Lock and Dam No. 2	Braddock, PA	1951	56	360	9	SP
		1953	110	720	9	SP
Lock and Dam No. 3	Elizabeth, PA	1907	56	360	8	SP
		1907	56	720	8	SP
Lock and Dam No. 4	Monessen, PA	1932	56	360	17	SP
		1932	56	720	17	SP
Maxwell Locks and Dam	Maxwell, PA	1965	84	720	20	BL2
		1965	84	720	20	BL2
Grays Landing Lock and Dam	Greensboro, PA	1993	84	720	15	SP
Lock and Dam No. 8	Point Marion, PA	1959	56	360	19	SP
Morgantown Lock and Dam	Morgantown, WV	1960	84	600	17	SP
Hildebrand Lock and Dam	Morgantown, WV	1960	84	600	21	SP
Opekiska Lock and Dam	Morgantown, WV	1964	84	600	22	SP
Ohio River						
Lock and Dam No. 53	Mound City, IL	1970	110	1,200	13	SPF
Lock and Dam No. 52	Brookport, IL	1972	110	1,200	12	SPF
Smithland Lock and Dam	Bolconda, IL	1980	110	1,200	22	SP
		1980	110	1,200	22	SP
Uniontown Locks and Dam	Uniontown, KY	1975	110	1,200	11	BL2
		1975	110	600	22	BL1
Newburg Locks and Dam	Newburg, IN	1975	110	1,200	16	BL2
		1975	110	600	16	BL1
Cannelton Locks and Dam	Cannelton, IN	1972	110	1,200	25	SP
		1972	110	600	25	BL1
McAlpine Locks and Dam	Louisville, KY	1961	110	1,200	37	BL2
		1921	110	600	37	SP
		1930	56	360	37	SP
Markland Locks and Dam	Markland, IN	1963	110	1,200	35	BL2
		1963	110	600	35	BL1
Capt. Anthony Meldahl Locks and Dam	Chilo, OH	1962	110	1,200	30	BL2
		1962	110	600	30	BL1

Project	Community in Vicinity	Year Opened	Width of Chamber (ft)	Locks Usable Length (ft)	Lift at Normal Pool (ft)	Filling System Acronym
Greenup Locks and	Greenup, KY	1962	110	1,200	30	BL2
Dam		1962	110	600	30	BL1
Robert C. Byrd Locks	Hogsett, WV	1993	110	1,200	23	SP
		1993	110	600	23	BL1
Racine Locks and Dam	Letart Falls, OH	1970	110	1,200	22	SP
		1970	110	600	22	BL1
Belleville Locks	Reedsville, OH	1969	110	1,200	22	BL2
and Dam		1969	110	600	22	BL1
Willow Island Locks and	Waverly, WV	1973	110	1,200	20	SP
Dam		1973	110	600	20	BL1
Hannibal Locks and	New Martinsville,	1972	110	1,200	21	SP
Dam	WV	1972	110	600	21	BL1
Pike Island Locks and	Warwood, WV	1965	110	1,200	18	SP
Dam		1965	110	600	18	BL1
New Cumberland	Stratton, OH	1961	110	1,200	21	SP
Locks and Dam		1961	110	600	21	BL1
Montgomery Island	Industry, PA	1936	110	600	18	SP
Locks and Dam		1936	56	360	18	SP
Dashields Locks and	Glenwillard, PA	1929	110	600	18	SP
Dam		1929	56	360	10	SP
Emsworth Locks and	Emsworth, PA	1921	110	600	18	SP
Dam		1921	56	360	18	SP
Okeechobee Waterway, FL						
St. Lucie Lock and Dam	Stuart, FL	1941	50	250	13	SG
Moore Haven Lock	Moore Haven, FL	1953	50	250	2	—
Ortona Lock and Dam	LaBelle, FL	1937	50	250	11	—
W.P. Franklin Lock and Control Structure	Fort Myers, FL	1965	56	400	3	—
Old River, LA						
Old River Lock	Simmesport, LA	1963	75	1,200	35	SP
Ouachita and Black Rivers, AR						
Jonesville Lock and Dam	Jonesville, LA	1972	84	600	30	SP
Columbia Lock and Dam	Columbia, LA	1972	84	600	18	SP
Falsenthal Lock and Dam	Falsenthal, AR	1984	84	600	10	SP
HK Thacher Lock and Dam	Calion, AR	1984	84	600	14	SP

Project	Community in Vicinity	Year Opened	Width of Chamber (ft)	Locks Usable Length (ft)	Lift at Normal Pool (ft)	Filling System Acronym
Pearl River, MS and LA						
Lock 1	Pearl River, LA	1951	65	310	17	LC
Lock 2	Bush , LA	1951	65	310	15	LC
Lock 3	Sun, LA	1951	65	310	11	LC
Red River, LA						
Lindy Claiborne Boggs Lock and Dam	Red River, LA	1987	84	785	36	SP
John H. Overton	—	1987	84	785	24	SP
Lock and Dam No. 3	—	1987	84	785	31	SP
Lock and Dam No. 4	—	1987	84	785	25	SP
Lock and Dam No. 5	—	1987	84	785	25	SP
Sacramento River, CA						
Barge Canal Lock	West Sacramento, CA	1961	86	600	4	SG
Snake River, WA						
Ice Harbor Lock and Dam	Pasco, WA	1962	86	665	100	SBLC
Lower Monumental Lock and Dam	Walla Walla, WA	1969	86	666	98	SBLC
Little Goose Lock and Dam	Dayton, WA	1970	86	668	98	SBLC
Lower Granite Lock and Dam	Almota, WA	1975	86	674	100	HB8
St. Marys River, MI						
South Canal MacArthur Lock	Sault Ste. Marie, MI	1943	80	800	22	—
Poe Lock	Sault, Ste. Marie, MI	1968	110	1,200	22	SP
North Canal Davis Lock	Sault Ste. Marie, MI	1914	80	1,350	22	OC
Sabin Lock	Sault Ste. Marie, MI	1919	80	1,350	22	OC
Savannah River, GA						
Savannah River Lock and Dam	Augusta, GA	1936	56	360	15	SP
Tennessee River, TN, AL, MS, and KY						
Kentucky Lock and Dam	Gilbertsville, KY	1944	110	600	56	—
Pickwick Landing Lock and Dam	Hamburg, TN	1937	110	600	55	—
		1984	110	1000	55	—
Wilson Lock and Dam						
Main Lock	Florence, AL	1959	110	600	94	VB8
Auxiliary Lock	Florence, AL	1927	60	292	47	SP

Project	Community in Vicinity	Year Opened	Width of Chamber (ft)	Locks Usable Length (ft)	Lift at Normal Pool (ft)	Filling System Acronym
General Joe Wheeler Lock and Dam						
Main Lock	Florence, Al	1963	110	600	48	SP
Auxiliary Lock	Florence, AL	1962	60	400	48	SP
Guntersville Lock and Dam						
Main Lock	Guntersville, AL	1965	110	600	39	SP
Auxiliary Lock	Guntersville, AL	1939	60	360	39	SP
Nickajack Lock and Dam	Chattanooga, TN	1967	110	600	39	SP
Chickamauga Lock and Dam	Chattanooga, TN	1940	60	360	49	SP
Watts Bar Lock and Dam	Breedenton, TN	1942	60	360	58	SP
Fort Loudon Lock and Dam	Lenoir City, TN	1943	60	360	72	MP
Melton Hill Lock and Dam (Clinch River)	Kingston, TN	1963	75	400	54	MP
Willamette River at Willamette Falls, OR						
Lock No. 1	Oregon City, OR	1872	37	175	10	VG
Lock No. 2	Oregon City, OR	1872	37	175	10	VG
Lock No. 3	Oregon City, OR	1872	37	175	10	VG
Lock No. 4	Oregon City, OR	1872	37	175	10	VG
Guard Lock	Oregon City, OR	1872	38	175	—	VG

This page intentionally left blank

INDEX