CECW-EH-D Engineer Manual 1110-2-1605	Department of the Army U.S. Army Corps of Engineers Washington, DC 20314-1000	EM 1110-2-1605 12 May 1987
	Engineering and Design HYDRAULIC DESIGN OF NAVIGATION DAMS	
	Distribution Restriction Statement Approved for public release; distribution is unlimited.	

EM 1110-2-1605
12 May 1987

US Army Corps
of Engineers

Hydraulic Design of Navigation Dams

ENGINEER MANUAL

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

DAEM-ECE-HD

Engineer Manual
No. 1110-2-1605

12 May 1987

Engineering and Design
HYDRAULIC DESIGN OF NAVIGATION DAMS

1. Purpose. This manual provides current guidance and engineering procedures for the hydraulic design of navigation dams.

2. Applicability. This manual applies to all HQUSACE/OCE elements and field operating activities (FOA) having responsibility for the design of civil works projects.

3. General. Subjects covered are design, construction, and operation of navigation dams. The goal of a good design is to provide a cost effective structure with consideration given to social and environmental impacts.

FOR THE COMMANDER:

Arthur E. Williams
ARTHUR E. WILLIAMS
Colonel, Corps of Engineers
Chief of Staff

This manual supersedes EM 1110-2-2606 dated June 1952

DAEN–ECE–HD

DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 2031 4-1000

EM 1110-2-1605
12 May 87

Engineer Manual
No. 1110–2–1605

Engineering and Design
HYDRAULIC DESIGN OF NAVIGATION DAMS

Table of Contents

CHAPTER 1

INTRODUCTION

Section I. General

1-1. Purpose. This manual provides guidance in designing, constructing, and operating navigation dams. Some of the factors affecting the safety and efficiency of waterways that are discussed include: types of dams; environmental considerations; equipment in general use on navigation dams; options of design to accommodate ice/debris passage, emergency operation; normal operation to pass flood flows, removal of sediment, or assistance in hydropower development. Some information is also provided on the repair and rehabilitation of existing structures.

1-2. Applicability. This manual applies to all HQ-USACE/OCE elements and all field operating activities having responsibilities for the design of civil works projects.

1-3. References.

a. National Environmental Policy Act (NEPA), PL 9-190, Section 102(2)(c), 1 Jan 1970, 83 Stat 853.

b. ER 1110-2-50, Low Level Discharge Facilities for Drawdown of Impoundments.

c. ER 1110-2-1403, Hydraulic and Hydrologic Studies by Corps Separate Field Operating Activities and Others.

d. ER 1110-2-1458, Hydraulic Design of Shallow Draft Navigation Projects.

e. EM 1110-2-1405, Flood Hydrograph Analysis and Computation.

f. EM 1110-2-1408, Routing of Floods Through River Channels.

g. EM 1110-2-1409, Backwater Curves in River Channels.

h. EM 1110-2-1411, Standard Project Flood Determinations.

i. EM 1110-2-1601, Hydraulic Design of Flood Control Channels.

j. EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works.

k. EM 1110-2-1603, Hydraulic Design of Spillways.

l. EM 1110-2-1604, Hydraulic Design of Navigation Locks.

m. EM 1110-2-1611, Layout and Design of Shallow Draft Waterways.

n. EM 1110-2-1612, Ice Engineering.

o. EM 1110-2-1901, Soil Mechanics Design Seepage Control.

p. EM 1110-2-2701, Vertical Lift Crest Gates.

q. EM 1110-2-2702, Design of Spillway Tainter Gates.

r. EM 1110-2-4000, Reservoir Sedimentation Investigations Programs.

s. Hydraulic Design Criteria (HDC) sheets and charts. Available from: Technical Information Center, US Army Engineer Waterways Experiment Station (WES), PO Box 631, Vicksburg, MS 39180-0631

1-4. Bibliography. Bibliographic items are indicated throughout the manual by' numbers (item I, 2, etc.) that correspond to similarly numbered items in Appendix A. They are available for loan by request to the Technical Information Center Library, US Army Engineer Waterways Experiment Station, PO Box 631, Vicksburg, MS 39180-0631.

1-5. Symbols. A list of symbols is included as Appendix B, and as far as practical, agrees with the American Standard Letter Symbols for Hydraulics (item I of Appendix A).

1-6. Other Guidance and Design Aids. Use has been made of the following:

a. Hydraulic Design Criteria (HDC).[s] This loose-leaf design notebook was prepared and is maintained by OCE and WES. References to these criteria are by specific HDC chart numbers. Since the charts are periodically updated, users need to verify the latest versions. Complete notebooks are available from: Technical Information Center, US Army Engineer Waterways Experiment Station (WES), PO Box 631, Vicksburg, MS 39180-0631.

b. Computer Program Library. The WES Computer Program Library (WESLIB) provides time-sharing computer services to CE Divisions and Districts. One such service is the Conversationally Oriented Real-Time Program-Generating System (CORPS) that especially provides the noncomputer-oriented or noncomputer-expert engineer a set of proven engineering application programs, which can be accessed on several different computer systems with little or no training. (Item 9 of Appendix A gives instructions on use of the system and a partial list of available programs. Updated lists of programs can be obtained through the CORPS system.)

c. Project Design Memorandums. Liberal use has been made of design memorandums and model study reports resulting from Corps District studies for specific projects. These references are used generally to illustrate a design concept rather than provide specific feature dimensions for proposed projects.

1-7. WES Capabilities and Services. WES has capabilities and furnishes services in the fields of hydraulic modeling, analysis, design, and prototype testing. Recently, expertise has been developed in the areas of water quality studies, mathematical modeling, and computer programming. Procedures necessary to arrange for WES participation in hydraulic studies of all types are covered in ER 1110-2-1403. WES also has the responsibility for coordinating the Corps of Engineers hydraulic prototype test program.

1-8. Design Memorandum Presentations. General and feature design memoranda should contain sufficient information to assure that the reviewer is able to reach an independent conclusion as to the design adequacy. For convenience, the hydraulic information, factors, studies, and logic used to establish such basic spillway features as type, location, alignment, elevation, size, and discharge should be summarized at the beginning of the hydraulic design section. Basic assumptions, equations, coefficients, alternative designs, consequences of flow exceeding the design flow, etc., should be complete and given in appropriate places in the hydraulic presentation. Operating characteristics and restrictions over the full range of potential discharge should be presented for all release facilities provided.

Section II. Typical Navigation Projects

1-9. Navigation Dams. The Corps of Engineers has built or operated 182 navigation dams. These dams have normal heads from one foot to over 100 feet. Most dams have spillways with either a gated or uncontrolled crest section. However, a few projects such as Bay Springs on the Tennessee-Tombigbee Waterway or Lock 2 on the Arkansas River System have no spillways; they are both located in canals which traverse two drainage basins. Their upper pools are controlled by spillways located on the main river for the drainage basin on the upstream end of the canal. An inventory of reports on navigation dams is provided in Appendix C. Inland waterway design studies are outlined in ER 1110-2-1458. Lock design procedures are found in EM 1110-2-1604.

1-10. Basic Project Components. Navigation dams 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 the following :

a. Spillway (gated or uncontrolled).

b. Overflow embankment or weir.

c. Nonoverflow embankment.

d. Navigation pass.

e. Lock or locks.

f. Out let works.

1-11. 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 passages facilities.

c. Recreation facilities.

d. Water supply intakes.

e. Water quality, low-flow controls, multilevel outlets.

f. Irrigation outlet works.

Section III. Special Considerations

1-12. Safety. The safety of the public is an important consideration in the design and operation of navigation dams. Many individuals do not recognize some of the dangerous situations that exist near hydraulic structures. Uneven gate operation can result in eddy action that can sweep small boats into the stilling basin. Overflow weirs form zones of reverse flow (Figure l-l) for certain tailwater conditions which have resulted in small boats being trapped and capsized by the roller action. Surges downstream of locks and hydropower installations can pose hazards to small boats. The hydraulic designer should have input into determining the limited public access areas downstream of structures.

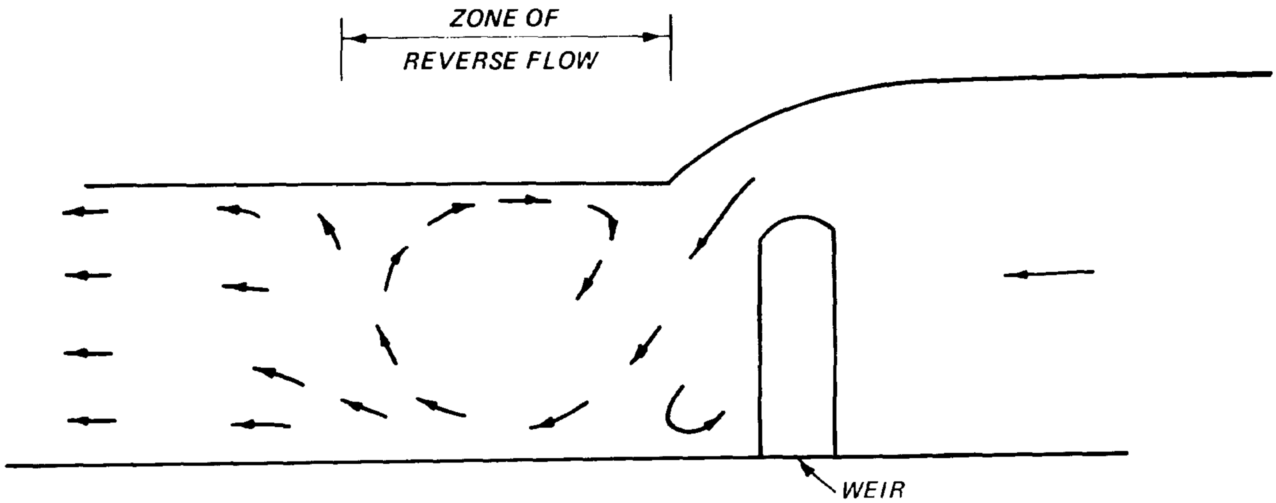


Figure 1-1. Reverse flow downstream of overflow weir

1-13. Environmental. Design of low-head navigation dams should consider measures prevent environmental degradation, as well as enhancement where possible. Design should also facilitate operational procedures for environmental enhancement. Opportunities to add enhancing features should be considered during planning and design. Water quality effects frequently cited for low-head navigation dams are low dissolved oxygen (DO) or nitrogen supersaturation. DO levels in a stream are increased in high turbulence in the presence of air, such as in a stilling basin. In some cases, stream reaeration can be enhanced by the mode of operation, such as proper gate operation during low-flow periods in the summer and fall when DO levels are typically lower. Aeration devices can be installed on the downstream face of the spillway to promote aeration. Nitrogen dissolved to supersaturation levels can be induced by operation of navigation dams particularly where there is a submerged hydraulic jump and low velocity in the downstream flow. This condition can stress aquatic life. During design, projects should be investigated for nitrogen supersaturation potential. An example of environment enhancement at a low-head navigation dam exists at McAlpine Locks and Dam, constructed at the falls of the Ohio River. These falls were historically a habitat for shore and wading birds. Modernization of McAlpine Dam reduced the flow over the rocks that provided feeding opportunities for the birds. Constructing low-overflow sections in the fixed-weir portion of the dam provided a relatively continuous flow that has improved the habitat measurably.

1-14. Aesthetics. Aesthetics should be given consideration by the designers of navigation structures. The size, shape, and composition of elements of the dam primarily are determined by functional requirements; however, as much as possible, the elements should be designed to be visually pleasing when combined with all other elements of the navigation structure. Some European projects have used streamlined piers with gate-operating mechanisms contained within the piers. This type of installation would provide an improved structure appearance as well as protection for gate-operating equipment. Another method of improving a structure’s appearance is the use of “pebble-finished” concrete surfaces as opposed to a smooth form-finished surface. The hydraulic designer should ensure that these surfaces are not used in areas of high-velocity flow.

CHAPTER 2

PROJECT IDENTIFICATION

Section I. Navigation Systems

2-1. General Considerations. During the design of an individual navigation dam project within a series of projects, one must consider the total navigation system. The total system of dams must be considered during preliminary site selection to establish the complete navigation layout. Navigation dams should be designed and located to provide for passage of flood flows and safe transit for all traffic expected to use the waterways when flow conditions permit. Other multipurpose functions such as irrigation and power may need to be accommodated. Site alternatives are usually considered and initial site selection determined in the early stages. Other disciplines (geotechnical, structural, etc.) should be involved in the site selection as in all major design decisions. The site selection is made by evaluating the physical characteristics of each potential site and making comparative estimates of costs and advantages for each site that would be adaptable to either a single or multipurpose project plan. Consideration must be given to whether one site has important hydraulic, foundational, operational, economic, or environmental advantages over other alternatives.

Section II. Project Purposes

2-2. General. Project purposes and their overall social, environmental, and economic effects greatly influence the hydraulic design of navigation dams. Optimization of the hydraulic design and operation requires an awareness by the designer of the reliability, accuracy, sensitivity, and possible variances of the data used. The ever-increasing importance of environmental considerations requires that the designer maintain close liaison with many disciplines to be sure environmental and other objectives are satisfied in the design, General project purposes and related design considerations are briefly discussed in the following paragraphs.

2-3. Purposes.

a. Navigation. Reservoirs that store water for subsequent release to downstream navigation usually discharge at lower capacity than flood-control reservoirs, but the need for close regulation of the flow is more important. The navigation season often coincides with the season of low rainfall, and close regulation aids in the conservation of water.

b. Flood Control. Flood control structures are designed for relatively large capacities where close regulation of flow is less important than are other requirements. When large discharges must be released under high heads, the design of gates, water passages, and energy dissipators should be carefully developed.

c. Irrigation. The gates or valves for controlling irrigation flows are often basically different from those used for flood control due to the necessity for close regulation and conservation of water in arid regions.

Irrigation discharge facilities are normally much smaller in size than flood regulation facilities. The irrigation outlet sometimes discharges into a canal or conduit rather than to the original riverbed. These canals or conduits are usually at a higher level than the bed of the stream.

d. Water Supply. Municipal water supply intakes are sometimes provided in dams built primarily for other purposes. Such problems as future water supply requirements and peak demands for a municipality or industry should be determined in cooperation with engineers representing local interests. Reliability of service and quality of water are of prime importance in water supply problems. Multiple intakes and control mechanisms are often installed to assure reliability, to enable the water to be drawn from any selected reservoir level to obtain water of a desired temperature, and/or to draw from a stratum relatively free from silt or algae or other undesirable contents. Ease of maintenance and repair without interruption of service is of primary importance. An emergency closure gate for priority use by the resident engineer is required for water supply conduits through the dam.

e. Power. Power plants are not within the scope of this manual. However, if power plants are to be located in the vicinity of the locks and/or dams, they should be designed so as not to cause conditions that are adverse to navigation or spillway operation such as adverse flow patterns in lock approaches, high pool-level fluctuations, or surge waves.

f. Low-Flow Requirements. Continuous low-flow releases are required at some dams to satisfy environmental objectives, water supply, downstream water rights, etc. To meet these requirements multilevel intakes, skimmer weirs, or other provisions must be incorporated separately or in combination with other functions of the navigation dam facility.

g. Multiple Purpose. Any number of purposes may be combined in one project. However, each added purpose will impact on project features and generally complicate project operational requirements.

Section III. Project Studies

2-4. General. The development of a navigation system involves a number of studies to determine the basic engineering feasibility of the proposed design. Study details are covered in later chapters, but a general discussion of some study purposes follows.

2-5. Project Water Requirements. Navigation projects require a minimum water supply for continuous operation. For projects with lock and dam structures, water supplies must be adequate to meet the following uses: lock filling and emptying needs (these depend on the proposed lock chamber size, lock lift, and maximum anticipated traffic); evaporation from the impounded pool; ground seepage from the pool; seepage under the dam; and leakage past spillway gates and other structural features. Minimum flows must also be adequate to meet the needs of other water-using project purposes such as water supply, irrigation, hydropower, environmental, etc. Procedures for evaluating minimum available flows are covered in Chapter 3.

2-6. Pool Levels. Navigation pool levels must be adequate to accommodate the drafts of design vessels plus the necessary clearances. Selected pool levels determine the dam classification. Dams with heads between 10 and 40 feet are generally considered low-head dams and those over 40 feet high-head dams. Because of the problems connected with overbank flooding, the pools of most navigation dams are essentially contained within the natural riverbanks, and would consequently be low-head projects. However, as explained in paragraph 3-11, navigation conditions are enhanced by the pool stability provided with high-head dams. If economically and environmentally feasible, high-head dams should be preferred.

2-7. Pool Storage. Inflow into navigation pools must always equal or exceed all outflows to ensure maintaining the navigation pool level. If natural minimum flows are inadequate to maintain the pools, flood storage should be provided to supplement natural flows during the low-flow periods. In an effort to minimize navigation pool fluctuations, the necessary storage should be provided in separate storage projects located in the drainage basin headwaters or on storage projects located on major tributaries.

2-8. Environmental. The general philosophy and guidance for preservation, mitigation, and/or enhancement of the natural environment have been set forth (item 33). Many scientific and engineering disciplines are involved in the environmental aspects of hydraulic structures. Some studies influencing the navigation dam design are briefly discussed below. Pertinent data from these studies should be presented in the design memorandum. The designers should have a working knowledge of these data and their limitations.

a. Fish and Wildlife. Navigation dam design and operation can maintain, enhance, or damage downstream fish and wildlife. Flow releases not compatible with naturally seasonal stream quantity and quality can drastically change aquatic life. These changes may be beneficial or may be damaging, such as adverse temperatures, chemical composition, or nitrogen supersaturation. Information from fish and wildlife specialists on the desired stream regimen should be obtained and considered in the design. Downstream wildlife requirements may fix minimum low-flow discharges. The water quality presentation should include summary data on requirements and reference to source studies.

b. Recreation. Recreation needs including fishing, camping, boating, and swimming facilities, scenic outlooks, etc., should be considered in the design of the project. These requirements are usually formulated by the planning discipline in cooperation with local interests. To accomplish the desired objectives, close cooperation between the hydraulic and planning engineers is required. Special consideration should be given to facilities for the handicapped, such as wheelchair ramps to fishing sites below stilling basins. Safety fences for the protection of facilities and the public are important. Appreciable damage to stilling basins has resulted from rocks thrown in by the public.

c. Water Quality. An awareness of maintaining and/or enhancing the environment within the past decade has brought into existence a relatively new and expanded art of reservoir hydrodynamics. Until recently, the study of reservoir hydrodynamics has been limited to a few prototype vertical temperature gradients and recognition of the seasonal inversions accompanying the fall surface water cooling. However, environmental considerations of today have necessitated the development of preproject capability for prediction of the expected seasonal reservoir stratification and circulation to permit construction and operation of navigation dams designed to meet storage and outflow regimes needed for the reservoir and downstream environment. Reservoir hydrodynamic studies may be done by other than the hydraulic designer (such as the hydrologic engineers) and they would specify the withdrawal requirements (quantity, elevation, etc.). The hydraulic engineer then designs the navigation dam to meet these requirements. However, the hydraulic designer furnishes some of the information for the hydrologic studies. The most common water quality parameter that needs consideration for low-head navigation dams is the downstream DO concentration. The reaeration of the discharge from these types of projects will need to be given consideration in design. Also see paragraph 1-13.

d. Environmental Impact Statements. Section 102(2)(c) of the National Environmental Policy Act (NEPA)[a] requires detailed documentation in the project design memoranda on the impact of the planned project on the environment. The hydraulic engineer may be required to cooperate in the preparation of impact statements. An analysis of 234 Corps of Engineers environmental impact statements on various projects is given in IWR Report No. 72-3 (item 31 of Appendix A). This report can be used as a guide to the type of material needed and format to be used in developing these statements. Basic to environmental impact statements are studies made to define the preproject and project functions and their effects on the environment. In most cases the effect of each project function must be set forth in detail. Item 12 of Appendix A summarizes the concepts involved and presents examples relative to water resources impact assessments. Presentation of the hydraulic design in design memoranda must identify environmental requirements and demonstrate how these are satisfied by the hydraulic facility.

2-9. Foundations. Foundation information of interest to the hydraulic designer includes: composition and depth of overburden, quality of underlying rock, and quality of exposed rock. In addition, sideslope stability is of considerable importance in the design of riprap protection. Tailwater stage change rates are required for bank stability design. Sufficient foundation data and/or reference to its source should be included in the hydraulic presentation to substantiate the energy dissipator and exit channel design.

CHAPTER 3

PROJECT PARAMETERS

Section I. Hydrology

3-1. General. Watershed hydrology is one of the first needs in developing a navigable waterway. The hydrologic conditions along the full waterway length will 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 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.

3-2. Basin Description. An understanding of certain physical features of a basin are necessary to properly evaluate the hydrologic and hydraulic functions. These physical features, as outlined below, 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 main-stem 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 water table 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.

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.

3-3. Hydrologic Data. The hydrologic studies for a river basin identify the discharges which 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.

a. 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, etc. 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.

b. 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, etc., and also project-related relocations and real estate requirements. Typical discharges used to determine the elevations include: the two percent duration flow, the 2-, 10-, 50- and 100-year interval flood flows, and the standard project flood (SPF).

c. 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 is presented in Figure 3-1.

3-4. 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:

a. 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 gages maintained within a specified state.

b. Climatological Records. In the United States, climatological data

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

such as precipitation, evaporation, wind speed, temperature, etc., 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.

3-5. 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. A number of effective models have been developed, but the one most commonly used is Computer Program 723-X6-L2010, "Flood Hydrograph Package." This model, commonly called HEC-1, was developed and is maintained by the Hydrologic Engineering Center (HEC) located in Davis, California. The three main steps in developing an HEC-1 model for a specific project site are summarized.

a. Rainfall-Runoff. The HEC-1 model needs to represent the rainfall-runoff relationship at any particular location in the basin. This relationship is based on developing one or more unit hydrographs for that location within the basin. EM 1110-2-1405 provides guidance in unit hydrograph development.

b. Routing and Combining. The HEC-1 model is then used to route runoff from the various parts of the basin and combine them to establish flow conditions at the project location. General guidance for flood routing is contained in EM 1110-2-1408.

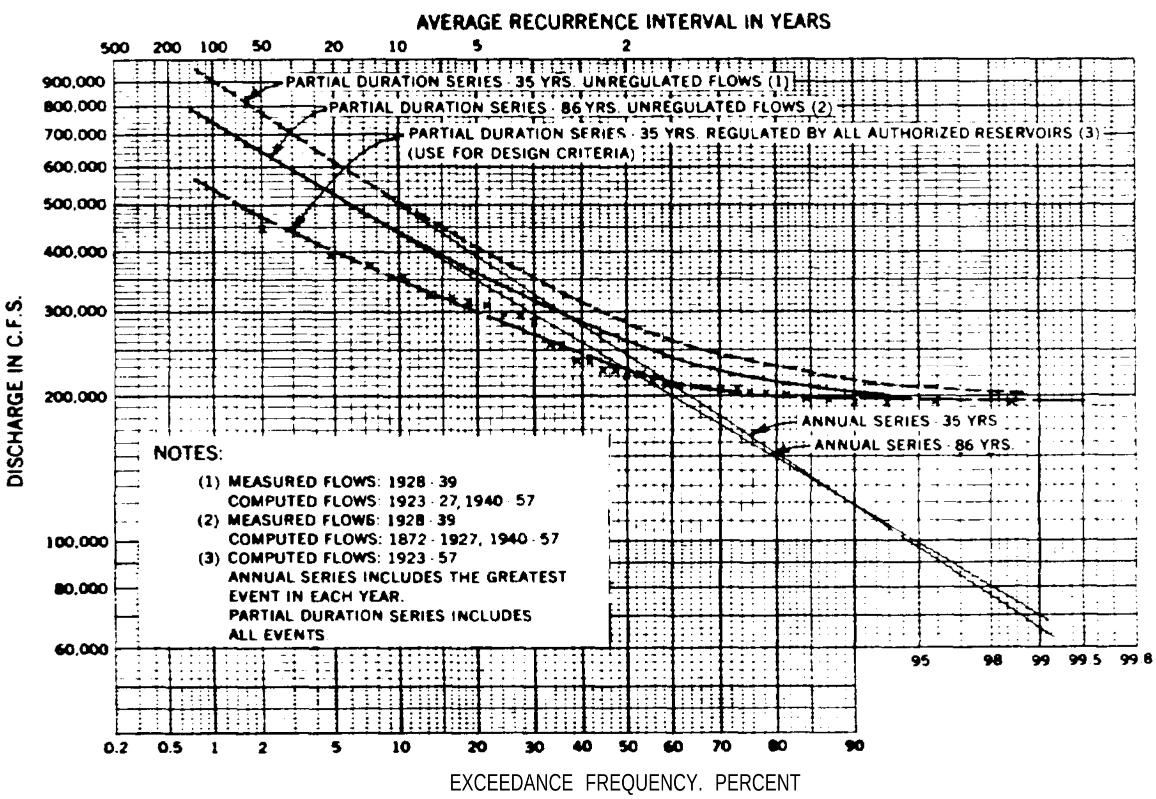
c. Calibration. Verification of the HEC-1 model requires an analysis of most of the experienced storms on the basin for which resulting flood hydrographs are known. Experienced rainfalls are applied to the model and the computed flood flows are compared with the experienced flood hydrographs. From several such tests, adjustments are made to the unit hydrographs, routing criteria, rainfall, and infiltration data within the model until a reasonable reproduction of all experienced flood hydrographs is obtained.

3-6. 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.

a. 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 (HMR's). For any particular project, the PMF discharges are determined by inputting the PMS rainfall data into the adopted HEC-1 model for the project.

b. <u>Standard Project Flood (SPF).</u> As identified in EM 1110-2-1411, 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. The EM provides the necessary guidance for developing this storm. For very large watersheds which are beyond the scope of EM 1110-2-1411, the SPS is frequently estimated to be half of the PMS as determined above.

c. <u>Flood Frequencies.</u> 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 percent chance of being equaled or exceeded in any one year. A typical discharge-frequency curve is shown in Figure 3-2.



LOCK AND DAM NO. 7
FREQUENCY OF PEAK DISCHARGES AT LITTLE ROCK

Figure 3-2. Typical discharge-frequency curve used in the design of Murray Lock and Dam, Arkansas River, Arkansas

d. Flow Duration. Lesser project flows are commonly expressed with respect to their duration--the percent 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. A typical stage-duration curve as derived from the discharge-duration and discharge rating curves for Murray Lock and Dam is shown in Figure 3-3.

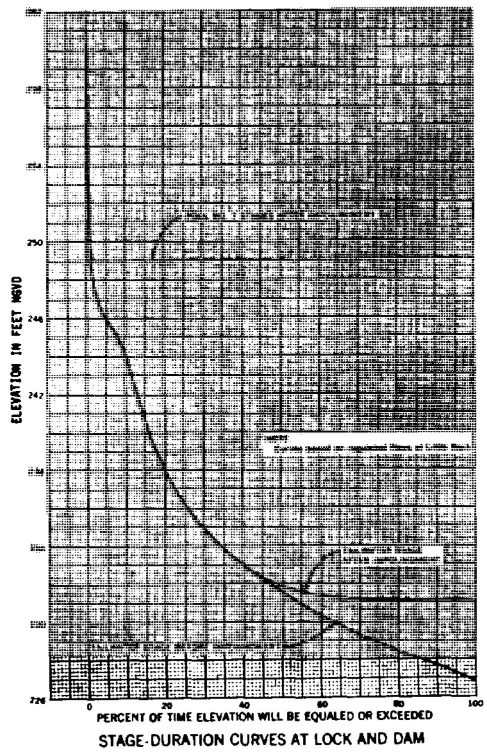


Figure 3-3. Typical stage-duration curve for Murray Lock and Dam, Arkansas River, Arkansas

Section II. Hydraulics

3-7. 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 postproject 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 assure satisfactory hydraulic performance. These second phase studies are covered in CHAPTER 5. The required stage-discharge studies presented here cover channel discharge rating curves, water-surface profiles, and establishment of navigation pool elevations.

3-8. 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-gaging stations. These stations are located at relatively fixed stream cross sections such as bridge openings or weir structures where the stage-discharge relationships will stay relatively fixed with time. Most existing stream-gaging stations are established and operated by the USGS. However, existing station locations are limited and establishing new stations for specific projects may be advisable.

a. 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. See paragraph 5-7 for factors that can affect tailwater rating curves.

b. 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.

3-9. 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.

a. 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. The most common computer program for conducting water-surface profiles is HEC-2. Information on the theoretical basis, latest version, and operating instructions for HEC-2 can be obtained from HEC, the model developer. Basic guidance for operating HEC-2 is provided in EM 1110-2-1409. Other satisfactory backwater programs have been developed for specific projects. For instance, the Arkansas River profiles were computed with the LRD-1 program. This program was developed for handling flood flows that spread over broad overbank areas containing both cleared and heavily wooded areas. However, the HEC-2 program is the most common one used to compute water-surface profiles.

b. Multiple Computations. During floods when water levels are well over the riverbanks, flow patterns can become very complex. Man-made obstructions such as transportation embankments, levees, building developments, dams, etc., 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. WES or HEC personnel can advise users on available hydrodynamic models.

c. Profile Plots. Plotting existing and postproject water-surface profiles over a complete navigation system can be a major undertaking. Many HEC-2 users have developed computer graphics programs for accomplishing this task. Information on many of these programs, both locally available and remotely accessible, can be obtained from HEC.

3-10. Specific Profile Uses. Following are descriptions of some of the most common uses of water-surface profiles in navigation dam design.

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

b. 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.

c. 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 lo-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 feet above the navigation pool or two feet above the IO-year recurrence interval flood. Access roads were set at the lo-year recurrence interval flood. Other feature elevations were similarly dependent on the profiles.

d. 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 loose their efficiency by reduction of the hydraulic heads between the fields or ditches and the river. A special study of water table 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. Study results are summarized in item 29.

3-11. 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.

a. 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 do 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.

b. 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 which 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 which 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.

Section III. Sedimentation

3-12. General, Sedimentation problems should be grouped into two main categories: (a) local scour and deposition problems and (b) general degradation and aggradation problems. The first is controlled or influenced primarily by the hydraulic design of the project structures. The second is the result of the stream's response to changes in the discharge hydrograph and sediment transport caused by the proposed navigation projects. Each of these problem areas should be reported separately. State the refinements, if any, for subsequent sedimentation studies and the impact of either more or less sediment on project performance. General information and guidance about sedimentation is obtained in EM 1110-2-4000.

3-13. Problems.

a. Alluvial rivers tend to establish an equilibrium between the water and sediment loads imposed upon them. Any significant modifications to the system (realignments, lock and 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.

b. Sediment problems are generally more difficult to predict for low-head 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.

3-14. 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.

3-15. 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 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 of and interpretation of results from sediment transport models.

3-16. 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 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 paragraph 3-15 of this section. 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 selection of model to be used, study limits for the model, and estimating 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 tools listed below, 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.

a. The first model the engineer should consider for analyzing sedimentation problems is “Scour and Deposition in Rivers and Reservoirs,” (HEC-6) developed by the HEC (item 28). HEC-6 is a one-dimensional flow model that can be used to analyze scour and deposition in both rivers and reservoirs. The program is very useful in determining long-term trends of scour and deposition in a stream channel and can be used to determine degradation that can be expected downstream of dams. Downstream degradation of the channel bed can be a significant problem in areas downstream of high-lift locks and dams. Deposition in navigation channels and lock approaches is usually a problem in low-lift and run of river projects. HEC-6 is useful in the initial studies associated with navigation project because of its ability to provide the location and volumes of deposition that can be expected with a navigation project. Locations and volumes of deposition can be used to estimate the amount of maintenance dredging that can be expected. Although one-dimensional models will point out locations and volumes of deposition, more detailed physical models and/or two-dimensional numerical model studies will most likely be needed to develop alternative methods of reducing or eliminating maintenance dredging. HEC-6 can also be used to study sedimentation problems that can be expected during floods and the effect dredging depth has on the rate of deposition. Detailed discussion of the input data for HEC-6 can be found in the user manual for HEC-6 and can be obtained from the HEC; briefly the data needs are geometric, sediment, hydrologic, and operational data. Of the models to be discussed in this section, HEC-6 will usually be most useful in the initial studies of the proposed system and are the only models that can address the entire system at one time, HEC-6 is not designed to model hydraulic structures in great detail and the user should not try to use HEC-6 in areas where the one-dimensional flow assumptions do not apply.

b. If it is determined that HEC-6 cannot adequately provide solutions to sediment problems, the TABS-2 modeling system can be used (item 25). A word of caution at this time is necessary in that when you decide to apply the TABS-2 system, everything involved gets bigger. The data required to do the modeling increase, the computer cost increases, and the level of expertise required to apply the model increases. TABS-2 is a generalized numerical modeling system for open-channel flows, sedimentation, and constituent transport. It consists of more than 40 computer programs to perform modeling and related tasks. The major modeling components--RMA-2V, STUDH, and RMA-4--calculate two-dimensional, depth-averaged flows; sedimentation; and dispersive transport, respectively. The other programs in the system perform digitizing, mesh generation, data management, graphical display, output analysis, and model interfacing tasks. Utilities include file management and automatic generation of computer job control instructions. TABS-2 has been applied to a variety of waterways, including rivers, estuaries, bays, and marshes. The TABS-2 model can be used to analyze scour and deposition problems associated with navigation structures, locks and dams, dikes, and approach and exit channels. TABS-2 is also a useful tool in lock site studies. If there are a number of possible sites to place a proposed lock and dam, the TABS-2 system can be used to determine the possible scour and deposition problems associated with each site and to evaluate preventive measures necessary to prevent sediment problems. Because of cost and data requirements, the TABS-2 model limits should be limited to area of concern and not used to model long reaches of river. Long reaches of river can be modeled more efficiently using HEC-6. The TABS-2 model is also a useful tool in the initial analysis of alternative methods of reducing sediment problems before construction and testing of physical models. Other sediment models are available, one of which is a stream tube model used to determine scour and fill trends in an alluvial stream. St. Louis District has applied the model to navigation dams, cofferdams, and other related structures,

c. 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 with the total engineering design and construction cost of a navigation project, and results of 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.

3-17. 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 involve 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 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. Bank stabilization methods can be found in numerous reports and design documents for the Arkansas and Red Rivers and good literary review can be found in Section 32 Bank Stabilization Report (Item 26). 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.

Section IV. Ice Conditions

3-18. 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. EM 1110-2-1612 provides methods of estimating ice growth and duration using winter air temperatures.

CHAPTER 4

PROJECT LAYOUT (SITING OF STRUCTURES)

4-1. General. Detailed guidance on project layout is provided in EM 1110-2-1611. This chapter provides an overview of the major aspects of project layout. Navigation locks and dams are usually required in some streams or canals to provide adequate depths for navigation during low flows without excessive velocities. Lock and dam layout is an iterative process in which the physical, hydraulic, geologic, and other parameters are evaluated for a preliminary layout and necessary adjustments made eventually come up with a feasible layout. The basic tools required to initiate this process are topographic maps, water-surface profiles, geological data, and preferably aerial mosaics that cover the full length of the proposed project. The number of navigation structures required for the system is basically dependent on the stream bed slope and on the levels of the upper pools that can be economically established. Ideally, the pools would be as high as possible to reduce the total number of lock and dam structures, thereby minimizing system transit time. Also, all the pools would have roughly equal heads so that lockage water requirements and operation times at each project are roughly the same. Physical constraints normally prevent attaining these ideal conditions.

4-2. Upper Pool Elevation. The selection of the optimum upper pool elevation will require a detailed analysis of the local terrain; areas subject to flooding; effects on groundwater elevation, drainage, environmental impact; need for raising, relocation, or replacement of existing facilities such as bridges, levees, highways, railroads, sewer lines, etc.; real estate acquisition; and need for dredging and/or training and stabilization structures. In some cases it might be more economical to increase the length of the pool by dredging in the upper reach than by raising the pool elevation. When sediment movement is involved training structures might be required to maintain navigation widths and depths in the dredged area. The ultimate selection of the upper pool elevation and location of the structures has to be based on an economic evaluation of the factors involved and navigation conditions that could result from the proposed project. However, navigation conditions are normally better with high-head pools because velocities are lower and pool fluctuations are less.

4-3. Navigation Considerations. The site selected for each structure can be one of the most important factors in the development of satisfactory navigation conditions. In addition to other factors, the design engineer should consider the reach upstream and downstream of the proposed sites (including current directions and velocities), sediment movement for the various flows possible, effects of the structure on the currents and movement of sediment, and the effects of the resulting currents on the movement of tows approaching and leaving the lock or locks.

4-4. Foundations. The foundation available may have a significant effect on the location and arrangement of the structure. 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 may reduce the number of sites available. Movement in the location of the structures because of foundation conditions should not jeopardize safe and efficient navigation conditions.

4-5. Sediment Movement. The effects of sediment movement in a stream should be considered in the evaluation of the location of the structures and the selection of the normal pool elevation. Many of these streams will require some modifications within the pool because of short-radius bends and shoaling in crossings, particularly in the upper reach of the pool. Solution of sedimentation problems requires a knowledge of the sedimentation processes in alluvial streams and methods that can be used to modify these processes to eliminate any undesirable conditions. Heavy sediment movement could have an effect on the length of pool that could be economically developed but would otherwise have little effect on the location of the individual structure. Normally, lock or locks sited to provide good navigation conditions (normally on the outside of bends) should experience no difficulties with shoaling in the upper lock approach. Shoaling can be expected in the lower lock approach because of the sudden expansion in channel width at the end of the riverside lock wall. However, structures have been developed in model studies that can be used to minimize or even eliminate shoaling in the lower approach and thereby minimize or eliminate maintenance costs (see EM 1110-2-1611).

4-6. Channel Rectification. The natural bends in most streams are too sharp and long to establish a safe navigation sailing line for most commercially sized tows. Lock and dam layout needs to consider these channel realignments to be assured of satisfactory approach conditions. Channel realignments should change the natural alignment as little as possible to minimize the changes to the natural flow regime of the stream. Severe regime changes frequently result in the stream not conforming to the proposed alignment. Expensive maintenance problems will result.

4-7. Channel Stabilization. Channel rectification measures normally require channel stabilization structures to coax the stream to assume the realignment. Channel stabilization structures are also useful in the control of sediment deposition tendencies. Through properly placed structures, ultimate channel maintenance dredging can be minimized.

CHAPTER 5

PROJECT DESIGN

Section I. Spillway Design

5-1. General. Navigation dams can be relatively high structures, such as those on the Columbia and Snake Rivers, in which cases the spillway should be designed in accordance with procedures described in EM 1110-2-1603. 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 5-1 and symbols are defined in Appendix B. An example design is provided at the end of this chapter.

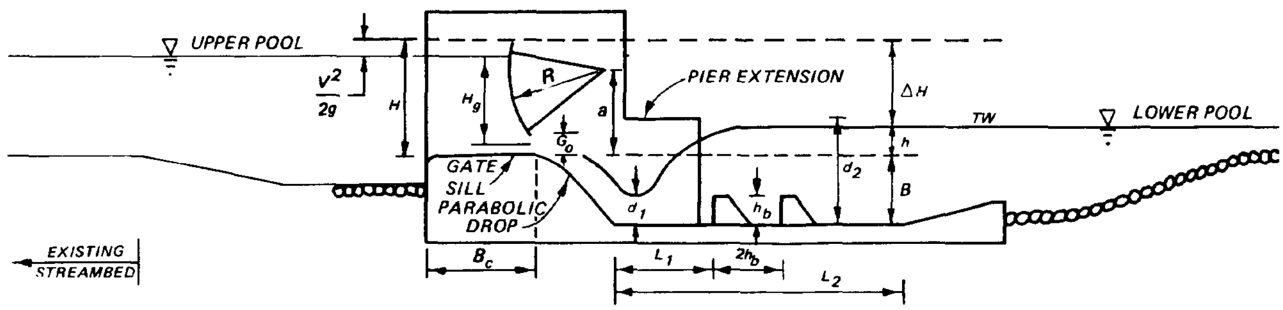


Figure 5-1. Definition sketch of typical navigation dam

5-2. Crest Design.

a. 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 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 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 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. For structures that do not operate under submerged flow conditions, an ogee crest is often used to improve efficiency of the spillway. EM-1110-2-1603 provides guidance for design of ogee crests. The remainder of paragraph 5-2 addresses crest design for broad-crested weirs.

b. 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. Based on an analysis of the data presented in item 3 of Appendix A, the minimum radius connecting the upstream face with the horizontal portion of the broad-crested weir should be as follows:

Head, feet	Radius, feet
<20	3
20-30	4
30-40	5
40-50	6

c. 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. In item 6 of Appendix A, tests are described wherein it was established that the downstream face of the weir should be parabolic based on the trajectory of a free jet, A free jet leaving the horizontal weir crest will follow the path:

$$X^2 = \frac{2V_o^2 Y}{g} \qquad (5\text{-}1)$$

where

X,Y = horizontal and vertical coordinates

V_o = 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

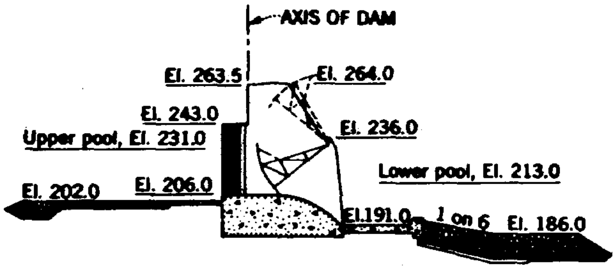
However, based on item 6 of Appendix A, the nappe will adhere to the downstream face if V is the theoretical velocity resulting from only one-third of the actual head. Thus, if the upper pool is 36 feet above the weir crest (H = 36 feet), V_o for determination of the shape of the downstream face of the weir should be based on a head of only 36/3 or 12 feet. That is, $V_o = \sqrt{2g(12)}$ = 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 feet, model testing is required. Using one-third of the head on the crest in Equation 5-1 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 feet. 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 (Item 6). Examples of different crests are shown in Figure 5-2. 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 feet) between upper and lower pool elevation.

d. Downstream Face, Submergible. Submergible tainter gates are used to pass ice over the top of the gates. As shown in Figure 5-3, 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 (c) above. Two examples of the downstream crest shape for the 2nd type of submergible tainter gate are shown in Figure 5-3. Gate bays for submergible gates should not be so wide that undesirable gate vibrations develop.

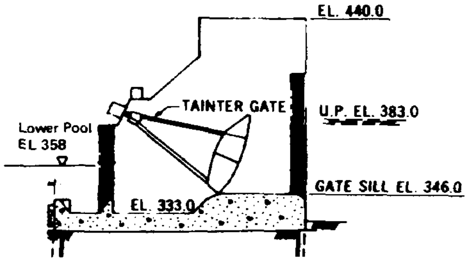
e. Intersection of Downstream Spillway Face and Stilling Basin Floor. Toe curves at the intersection of the downstream spillway face and the stilling basin floor are not widely used in low-head navigation dams. Guidance for toe curve pressures below ogee crests is given in HDC 122-5.

f. Crest Pressures, Velocities, and Water-Surface Profiles. For most low-head navigation dams, spillway velocities are relatively moderate because of tailwater submergence effects. Under normal spillway operations, all the gate openings would be balanced and maximum velocities would 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 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, etc. Available information is given in item 6 of Appendix A and shown in Figures 5-4 and 5-5 for water-surface profiles and pressures, respectively.

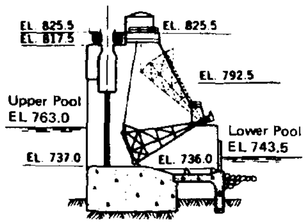
5-3. Spillway Capacity for High-Head Dams. Spillways for high-head navigation dams are generally designed with adequate capacity to pass the PMF flows. At this condition, all-flows-would still be limited to the spillway section; adjacent concrete or embankment structures would have adequate freeboard to



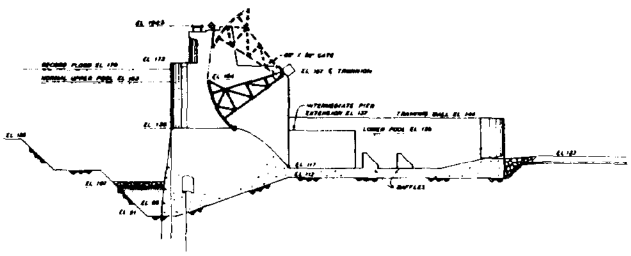
DAVID D. TERRY LOCK & DAM (NO.. 6)
(ARKANSAS RIVER)



CANNELTON LOCKS & DAM
(OHIO RIVER)

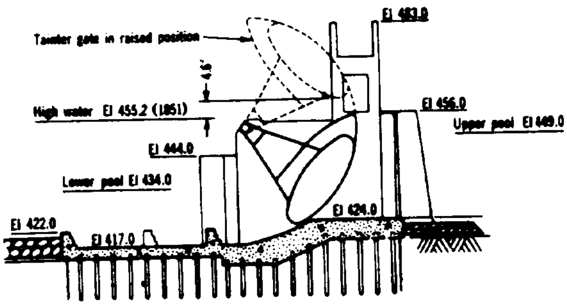


MAXWELL LOCK & DAM
(MONONGAHELA RIVER)

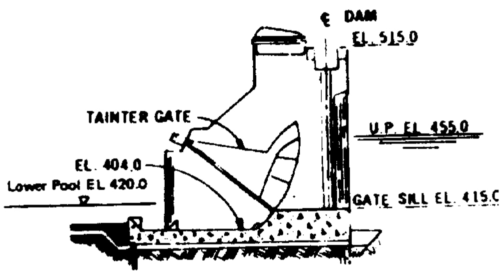


COLUMBUS LOCK & DAM
(TOMBIGBEE RIVER)

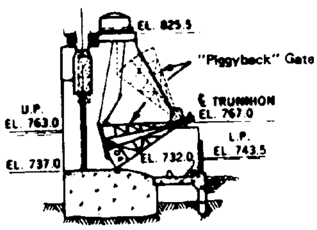
Figure 5-2. Examples of crests, nonsubmergible gates



LOCK & DAM 24
(MISSISSIPPI RIVER)



MARKLAND LOCKS & DAM
(OHIO RIVER)



MAXWELL LOCK & DAM
(MONONGAHELA RIVER)

Figure 5-3. Examples of crests, submergible gates

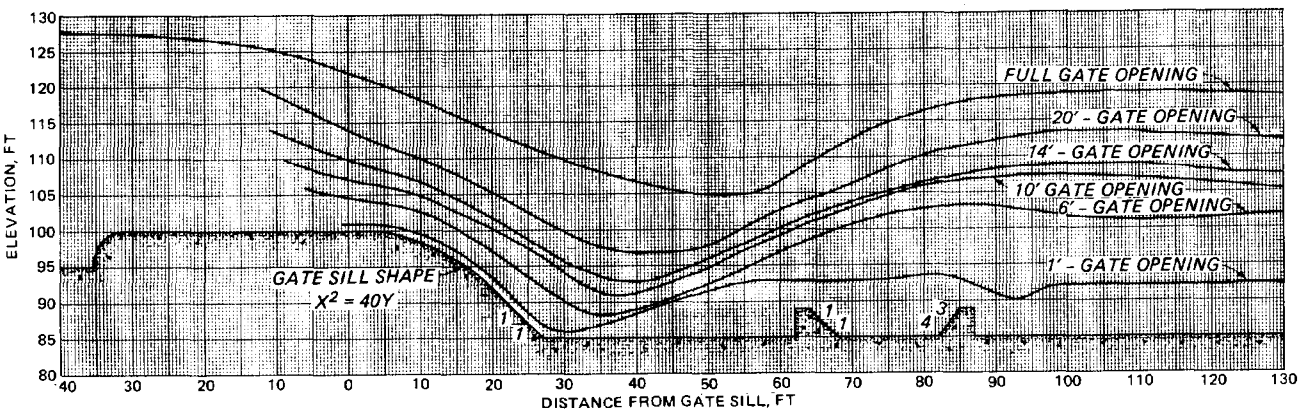


Figure 5-4. Water-surface profiles (from item 6, Appendix A)

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. Reference EM 1110-2-1603 for determining spillway capacity for high-head dams.

5-4. 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.

a. 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

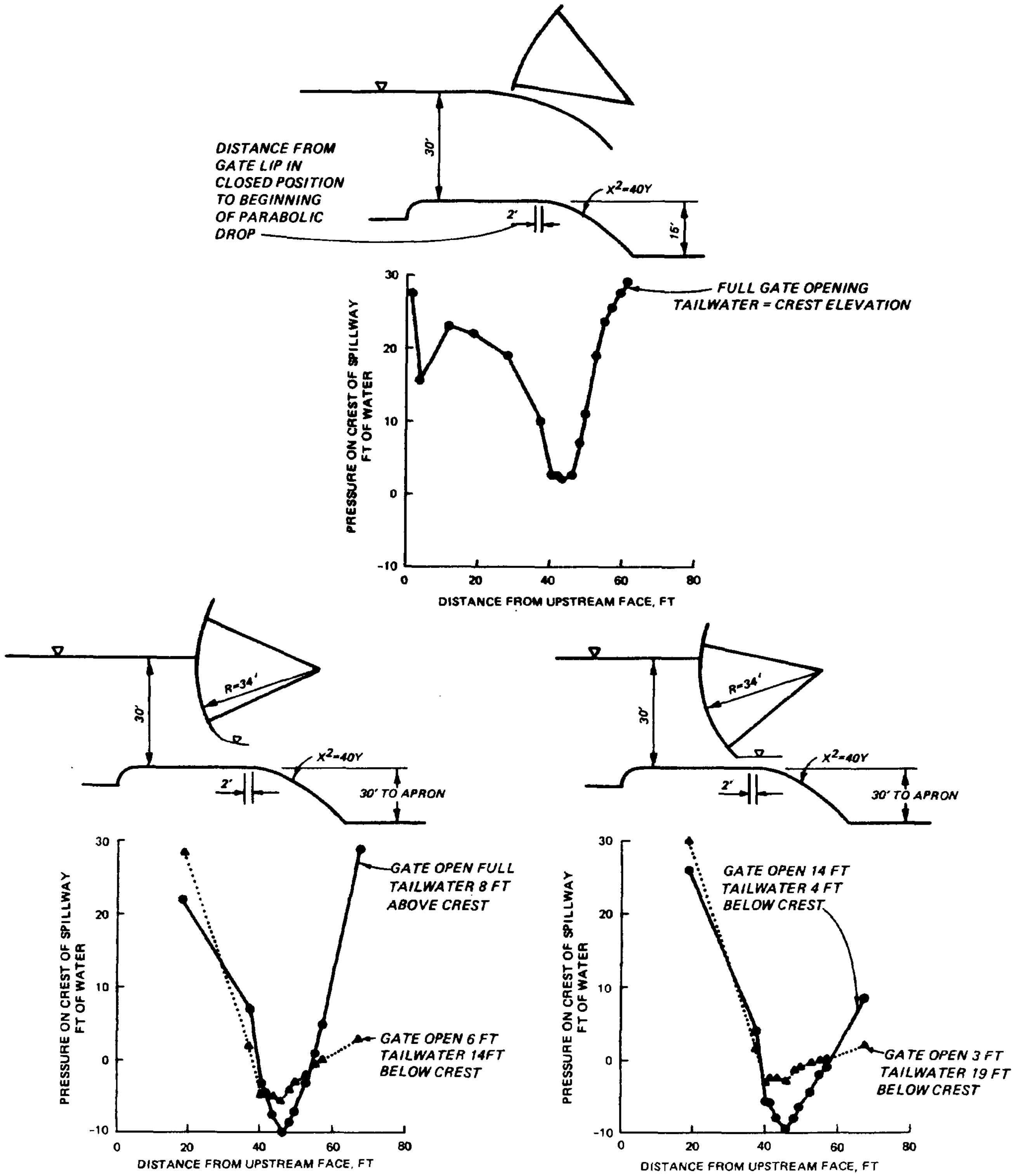


Figure 5-5. Pressures on crest for various gate openings (from item 6, Appendix A)

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 preproject 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.

b. 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.

5-5. Pool-Tailwater Relationships. The size of the spillway (both horizontal and vertical) affects pool and tailwater elevations. Three general cases can be identified.

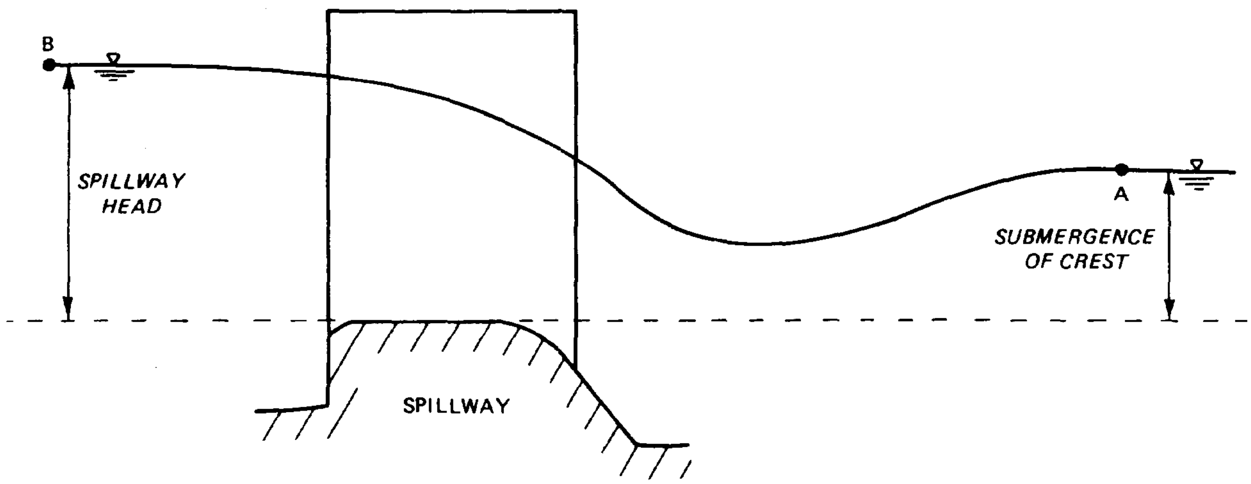
a. Case 1. The dam is of sufficient height that the spillway is not submerged by tailwater for any discharge.

b. 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.

c. 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 5-6) 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, etc. Determination of spillway design in relation to these factors is complex, but in general high, narrow spillways are spillway cost-effective, while 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.

5-6. 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 paragraph 5-7) or Kindsvater and Carter formulas can be used for an approximate pool elevation estimate during preliminary submerged spillway design studies (see item 32). 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 interval should also be computed to evaluate real estate, relocations, and other pertinent factors. Some Corps Districts have successfully used the special bridge routine in the HEC-2 backwater program to make these computations.



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

Figure 5-6. Spillway head/submergence

5-7. Discharge Rating Curves for Gated, Broad-Crested Weirs.

a. General. Discharge rating curves are needed for project design and operation. Low-head navigation structures have four possible regimes of flow that 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 5-7. Discharge coefficients for low-head navigation dams have been developed mainly for tainter gates. 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. Results from item 6 of Appendix A comparing the ogival and semicircular shapes showed no significant difference for the highly submerged broadcrested 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

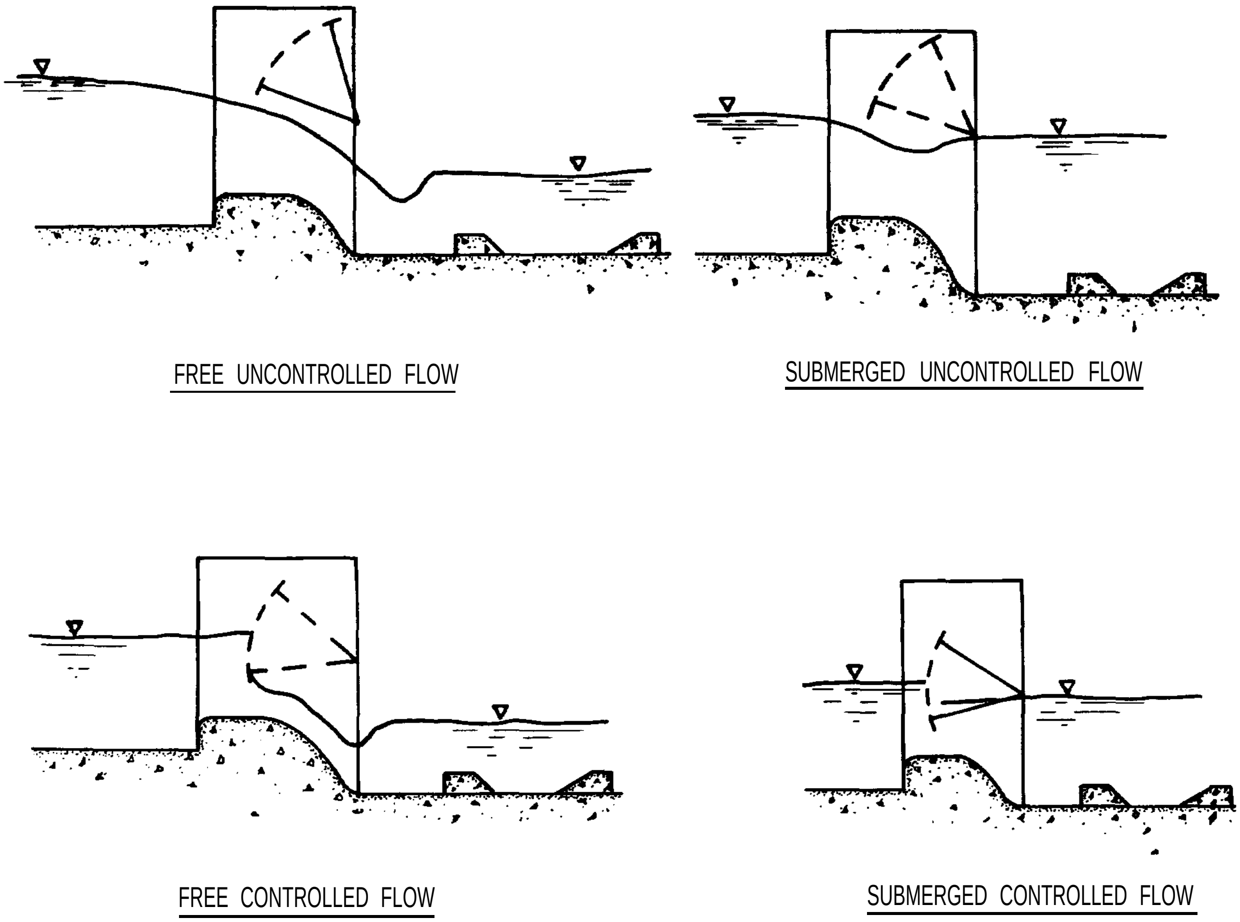


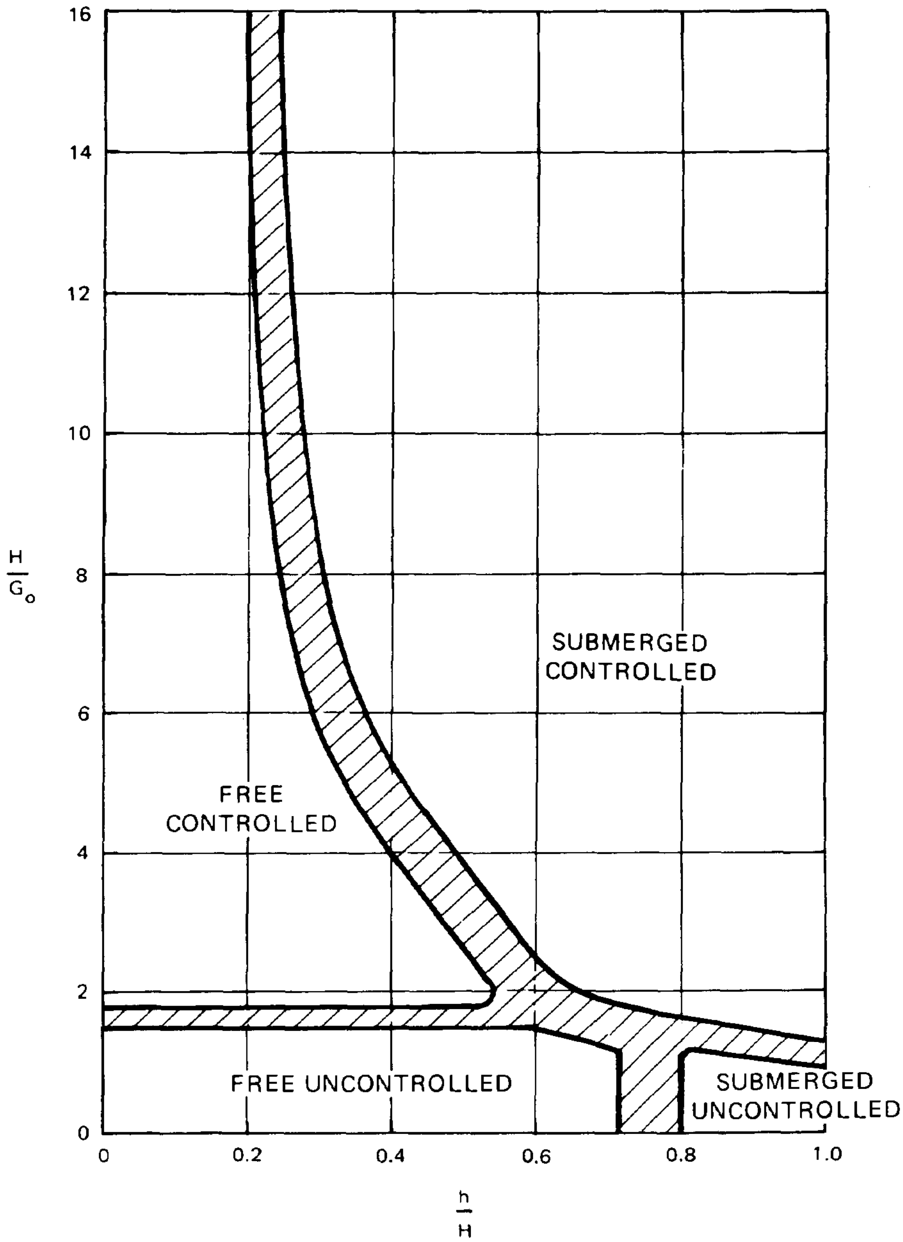
Figure 5-7. Four flow regimes

conditions are presented. A computer program was developed in the Pittsburgh District for discharge rating of navigation dams and is presented in item 22 of Appendix A.

b. Determining Flow Regime. Figure 5-8 gives guidance to determine the flow regime given headwater H, tailwater h, and gate opening G_o (definition sketch in Figure 5-1).

c. 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

$$Q = C_F L H^{3/2} \tag{5-2}$$



NOTE: CROSS-HATCHED AREAS REPRESENT TRANSITION ZONES
FULLY OPENED GATE EQUIVALENT TO $H/G_0 = 0$

Figure 5-8. Flow regime based on headwater, tailwater, and gate opening is applicable and free uncontrolled flow discharge coefficients versus (Head/Breadth of Crest) from item 22 of Appendix A are shown in 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

$$L_{effective} = L_{actual} - 2KH \tag{5-3}$$

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

d. <u>Submerged Uncontrolled Flow.</u> 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 :

$$Q = C_S LH^{3/2} \tag{5-4}$$

C_s from model data is shown to vary with h/H. Results from item 22 of Appendix A show that discharge coefficients for this flow regime are not significantly affected by stilling basin apron elevation. Figure 5-10 presents recommended submerged uncontrolled flow discharge coefficients as a function of h/H. These coefficients were developed from a large number of model investigations.

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

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

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 5-1

Suggested K values vary with spillway bay width as follows:

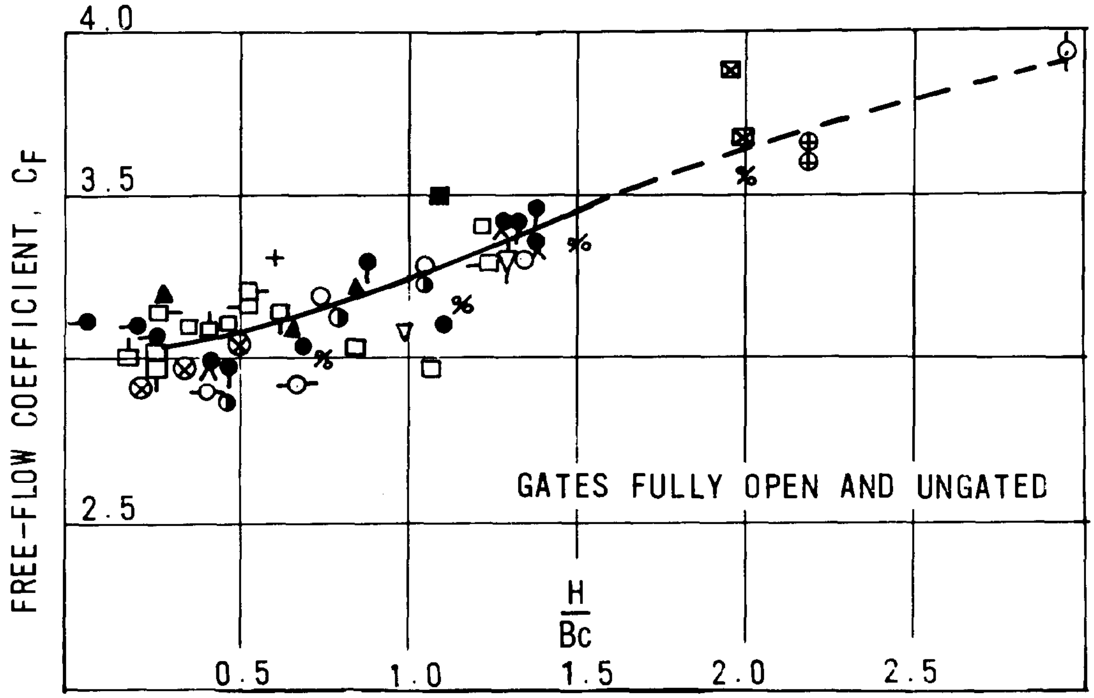


Figure 5-9. Free flow discharge coefficient for uncontrolled flow over a broad-crested weir

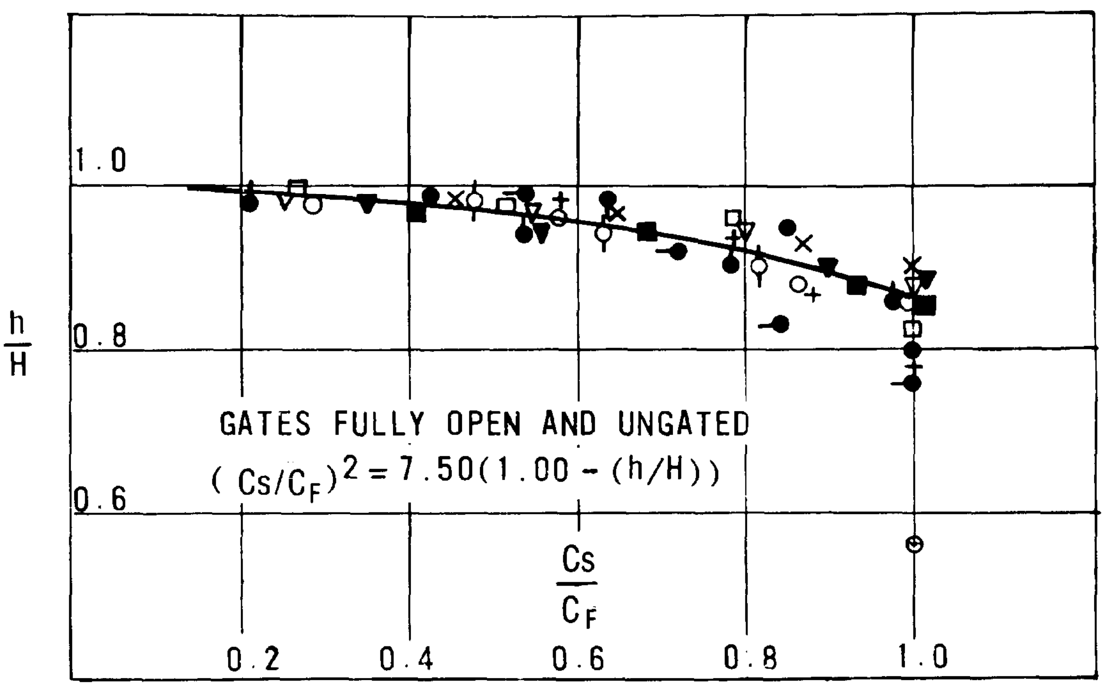


Figure 5-10. Submerged uncontrolled discharge coefficient for broad-crested weir

Bay Width, feet	K
40	0.80
50	0.85
60	0.90
110	0.95

These coefficients were developed from experience with prototype structures. Several different methods exist for predicting discharge for submerged uncontrolled flow. These include the methods presented above and HDC 111-4, items 6 and 32 in Appendix A. These methods do not give similar results.

e. 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) Results from item 22 of Appendix A shown in Figure 5-11 present the free controlled flow discharge coefficient as a function of gate opening, gate radius (R), trunnion height above crest (a), and gross head on the gate. Figure 5-11 is applicable to heads and gate openings less than 30 and 14 ft, respectively. The applicable equation is

$$Q = C_g L G_O \sqrt{2gH} \tag{5-6}$$

(2) For conditions outside the range covered in (1), 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.

f. 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

$$Q = C_{gs} L h \sqrt{2g\Delta H} \tag{5-7}$$

The submerged controlled discharge coefficient C_{gs} as a function of h/G for various apron elevations is given in Figure 5-12. See item 22 in Appendix A for a similar method for submerged controlled flow that has been used in the computer program referred in paragraph 5-7 (a).

g. Rating Curve Accuracy.

(1) Discharge Coefficients. Spillway rating curves as computed by the above 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

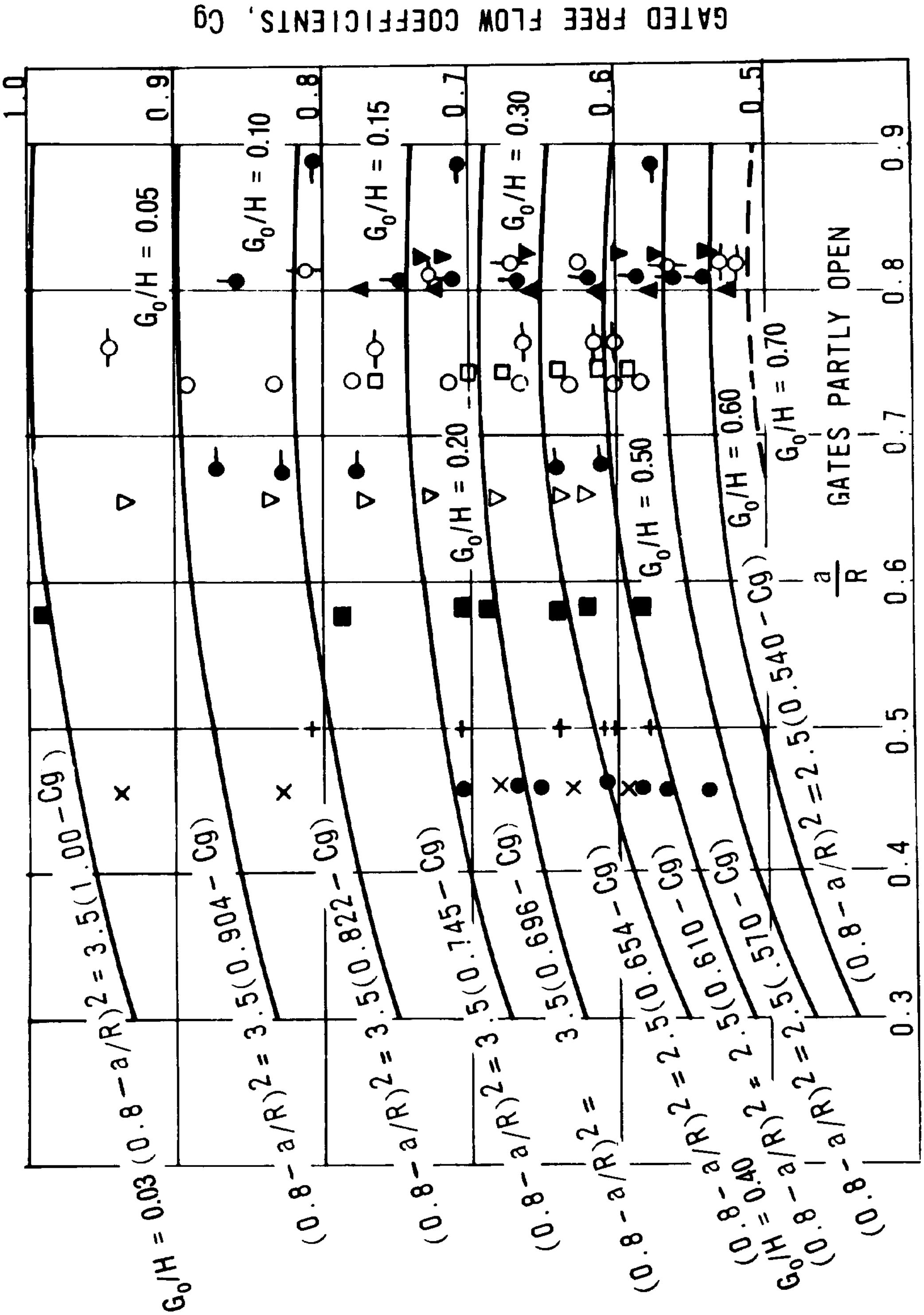
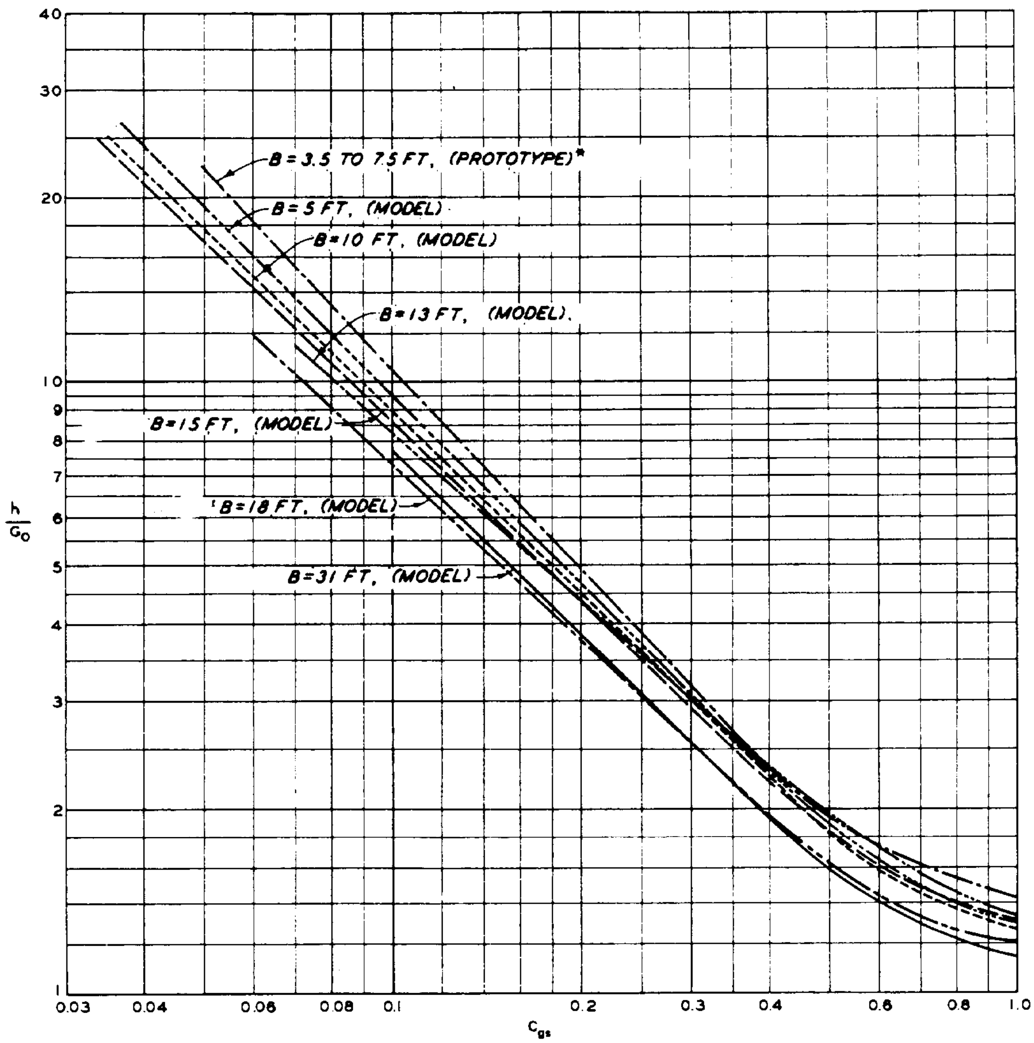


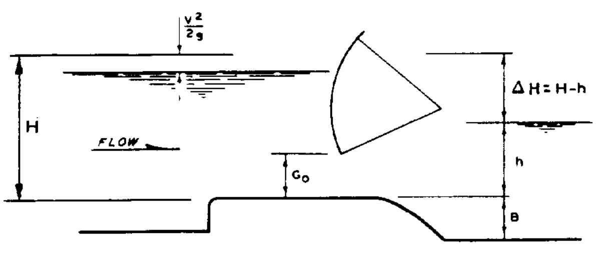
Figure 5-11. Discharge coefficients for free controlled flow
(from item 22, Appendix A)



BASIC EQUATION

$Q = C_{gs} \; Lh\sqrt{2g\,\Delta H}$

*MISSISSIPPI RIVER DAMS 2, 5A, AND 26



DEFINITION SKETCH

Figure 5-12. Discharge coefficients for submerged controlled flow (HDC 320-8) rating curve verification. In finalizing rating curves for major navigation systems, special prototype spillway measurements on similar existing projects should be considered.

(2) Tailwater Inaccuracies. Tailwater rating curves are extremely important to the design engineer. The selected tailwater curve will be used in 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 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 5-13. 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.

5-8. Overflow Embankments.

a. 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.

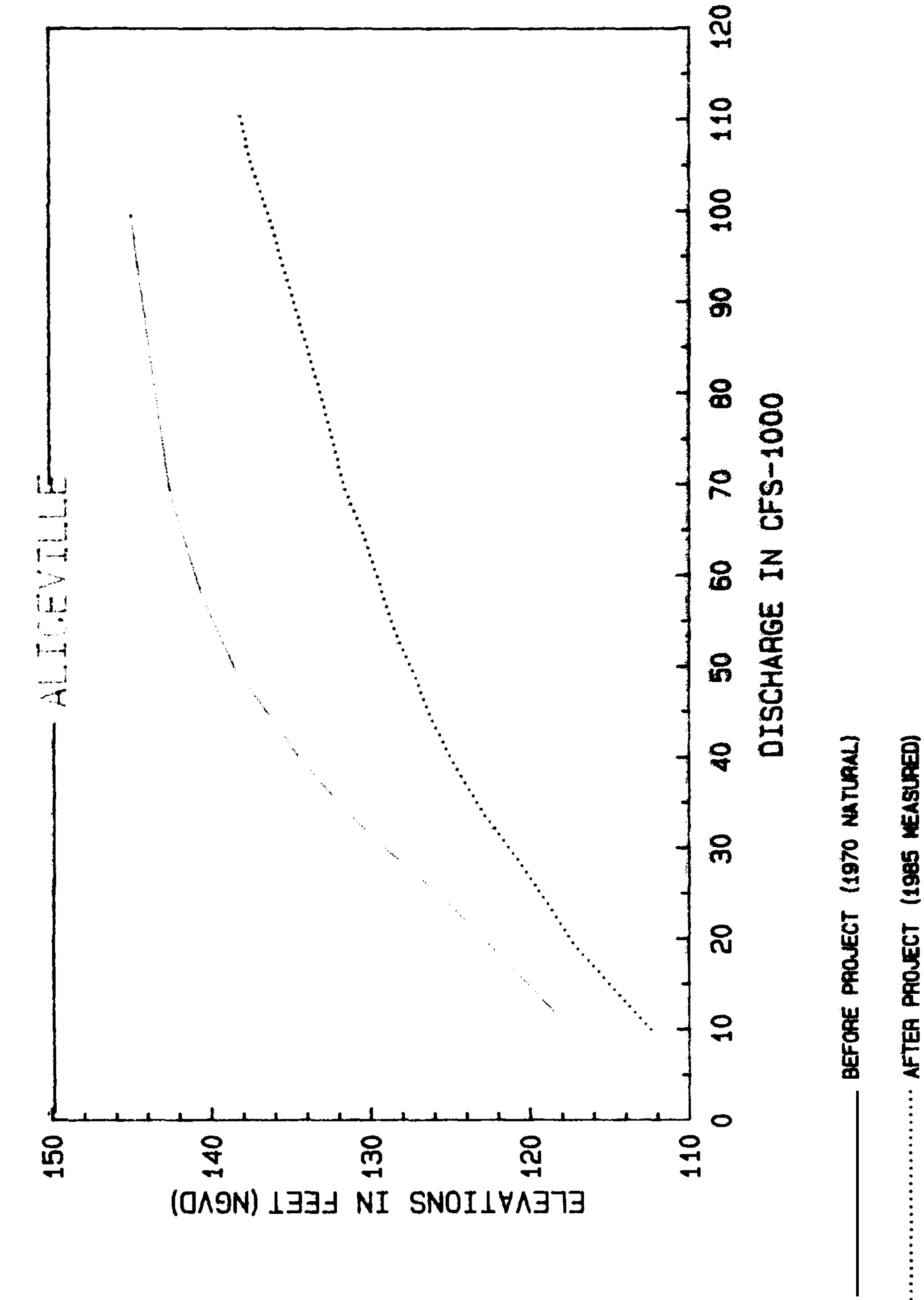


Figure 5-13. Tailwater rating curves, Aliceville Lock and Dam, Tennessee-Tombigbee Waterway

An example of an optimization study accomplishing this is given in Appendix D. 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 three feet. These riprap protected embankments are either access or nonaccess embankments having trapezoidal cross sections with a 1V-on-3H upstream face and a 1V-on-4H downstream face. The access embankments have a paved roadway on the crown of the embankment. Detailed discharge and riprap stability guidance is given in item 5 of Appendix A.

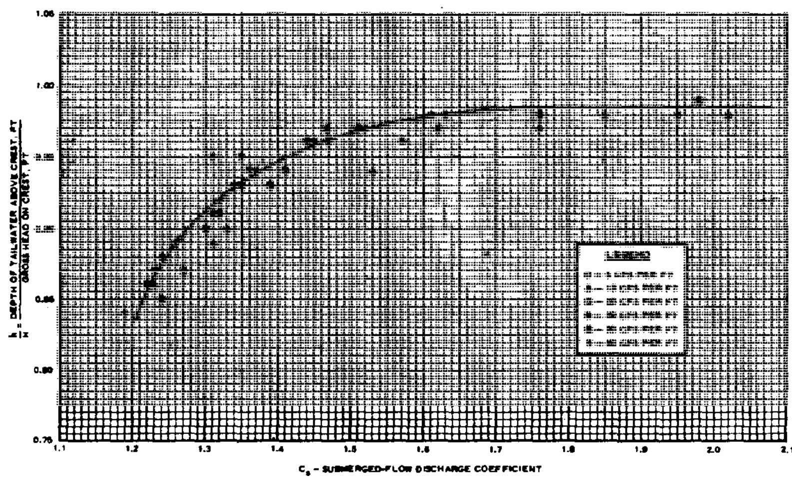
b. Discharge over Uncontrolled Sections. Figure 5-14 shows the submerged flow discharge coefficient for access and nonaccess type embankments. The second type of uncontrolled overflow section is the concrete wall having considerable height and designed to operate under submerged conditions. Discharge coefficients for a rectangular cross section and free flow conditions are shown in Figure 5-15; the reduction in free flow discharge due to submergence is also shown in Figure 5-15.

5-9. Stilling Basin Design.

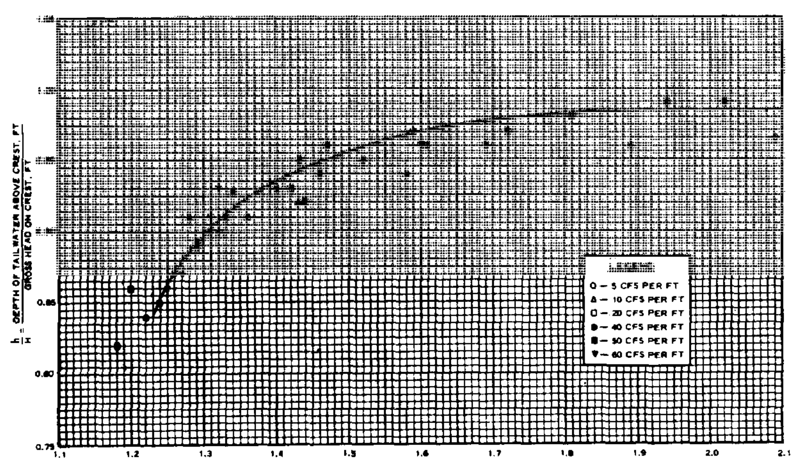
a. General. The purpose of the stilling basin is to reduce the kinetic energy of the flow entering the downstream exit channel. The stilling basin in conjunction with the downstream riprap ensures that local scour downstream of the structure will not undermine or otherwise threaten the integrity of the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

b. 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 (see items 15 and 18, Appendix A). 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.

c. Requirements for New Project Design. The following three 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.

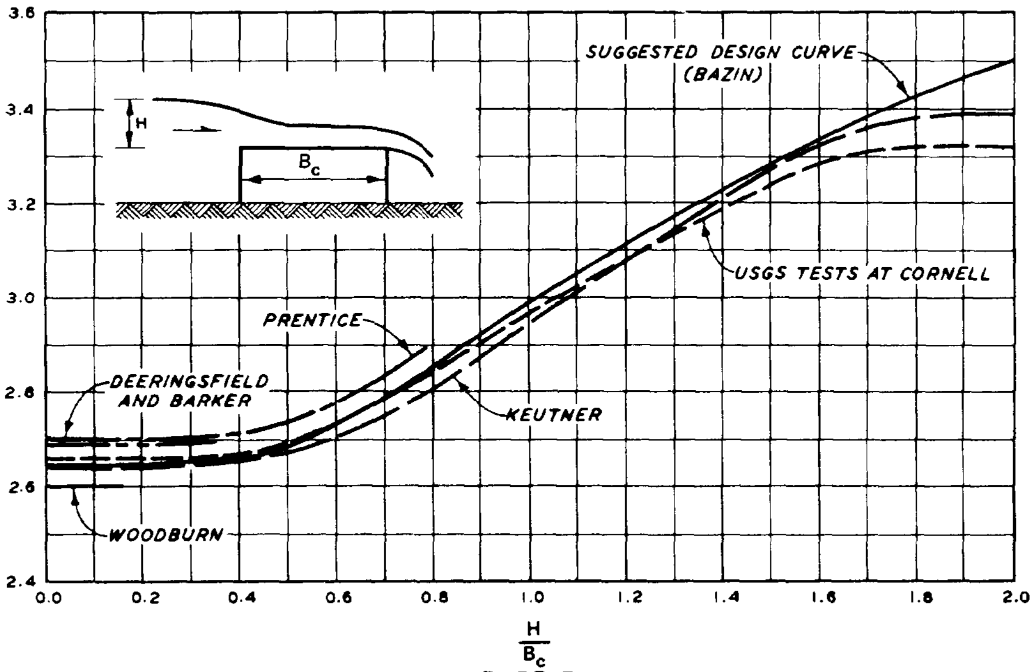


Access type embankments

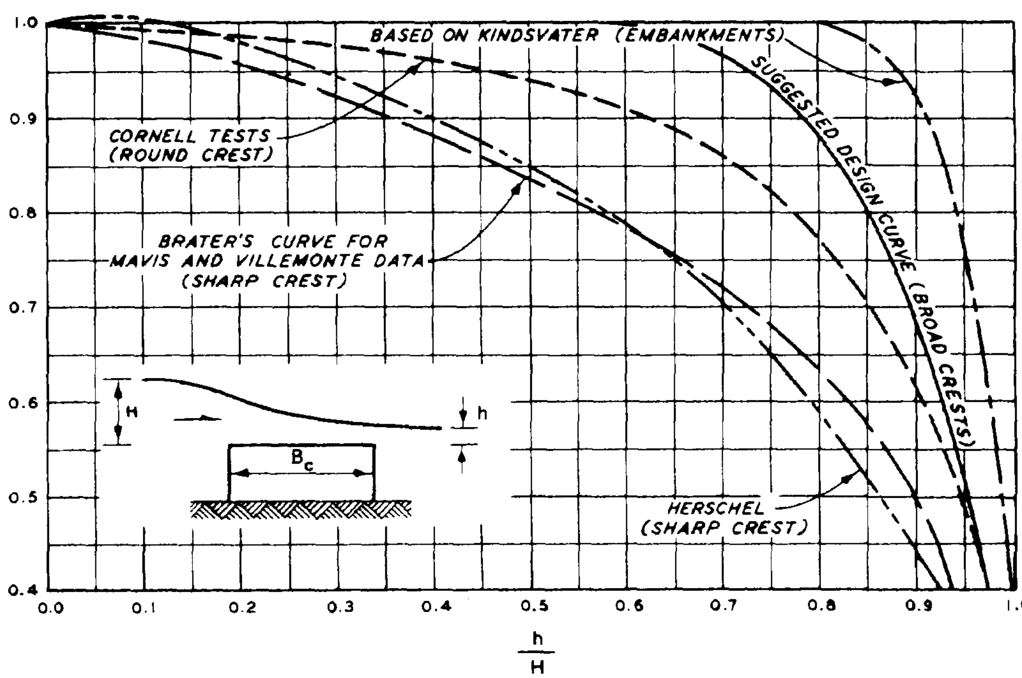


Nonaccess type embankments

Figure 5-14. Discharge coefficients for embankments under submerged flow (from item 5, Appendix A), $Q = C_s Lh \sqrt{2g\Delta H}$



a. FREE FLOW



b. SUBMERGED FLOW

NOTE : C_f = FREE-FLOW COEFFICIENT
C_s = SUBMERGED-FLOW COEFFICIENT
NEGLIGIBLE VELOCITY OF APPROACH

Figure 5-15. Low-monolith diversion, discharge coefficients (from HDC 711)

(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.

d. 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

$$\text{Upper Pool Elevation} + \text{Velocity Head Upstream} = \text{Stilling Basin Floor Elevation} + \frac{V_1^2}{2g} + d_1 \qquad (5\text{-}8)$$

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

$$F_1 = \frac{V_1}{\sqrt{gd_1}} \qquad (5\text{-}9)$$

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

$$\frac{d_2}{d_1} = 0.5\left(\sqrt{1 + 8F_1^2} - 1\right) \qquad (5\text{-}10)$$

(This form of the momentum equation ignores the forces on baffle blocks in the analysis. A comprehensive treatment of these forces in the momentum equation is given in item 2 of Appendix A.) At this point, the assumed stilling basin elevation is checked against the available tailwater according to

$$\begin{matrix}\text{Tailwater for} \\ \text{Given Discharge}\end{matrix} - \begin{matrix}\text{Assumed Stilling Basin} \\ \text{Floor Elevation}\end{matrix} = \text{Factor } (d_2) \qquad (5\text{-}11)$$

A new stilling basin floor elevation is assumed until Equation 5-11 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 5-11 can only be used in conjunction with Equation 5-10, the simplified momentum approach.

e. Recommendations from Results of Previous Model Tests.

(1) General. Model tests have been conducted at WES, Vicksburg, Miss. (items 10, 13-16 of Appendix A), during which stilling basin designs were developed for one gate half or fully opened. Recommendations from results of these tests are summarized in Table 5-1 and in the paragraphs that follow. 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--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 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 of the confining walls, usually is not a consideration.

(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 $0.85d_2$ for both one gate half or fully opened.

(3) Basin Length. Items 10 and 13-16 of Appendix A suggest a required length, L_2 from toe of jump to beginning of 1V-on-5H upslope of

$$L_2 = 2d_1F_1^{1.5} \qquad (5\text{-}12)$$

TABLE 5-1

Hydraulic Design Data from Physical Model Studies for Low-Head Navigation Dam Spillways Based on Single-Gate Opening (Fully and/or Half) Criteria

Project Name	Item No.	Basin No.	Designed for What Gate Opening	Unit Discharge cfs/ft	Bay Width ft	Entering Froude No. F'*	d_2 ft	$\frac{TW}{d_2}$	$\frac{L_1}{d_2}$	$\frac{\text{Baffle Height}}{d_2}$	$\frac{L_2}{d_2}$	$\frac{\text{End Sill Height}}{d_2}$	d_{50}† ft
L&D 26	15	16	Full	775	110	2.5	44.5	0.81	1.08	0.27	2.6	0.25	3.8
Aliceville	13	6	Full	350	60	3.5	30.5	0.72	1.31	0.26	2.6	0.16	2.2
Columbus	16	5	Full	350	60	3.7	31.0	0.77	1.29	0.26	2.6	0.16	2.0
Red River No. 1	14	16	Full	484	50	3.9	39.6	0.81	1.26	0.25	2.8	0.43	1.7
Red River No. 2	10	13	Full	683	60	2.4	39.5	0.71	1.06	0.23	2.5	0.18	2.5
Red River No. 3	**	2	Full	817	60	2.75	47.9	0.79	1.15	0.21	3.0	0.15	2.6
L&D 26	15	30	Ice and Debris	382	110	3.7	32.8	0.91	1.58	0.24	3.0	0.15	2.6
Columbus	16	4	Half	242	60	4.4	26.2	0.84	1.53	0.31	3.1	0.19	1.7
Red River No. 1	14	9	Half	390	50	3.6	33.2	0.72	1.51	0.33	3.0	0.09	
Red River No. 1	14	17	Half	370	50	3.7	32.3	0.74	1.55	0.32	3.1	0.09	1.5
Red River No. 1	14	7	Half	370	50	4.1	33.7	0.71	1.48	0.34	2.7	0.27	1.1

* Assumes no energy loss between upper pool and stilling basin.
** Unpublished draft report.
† d_{50} size immediately downstream from end sill.
Definition sketch shown in Figure 5-1.

(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 items 10 and 13-16 of Appendix A are as follows:

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

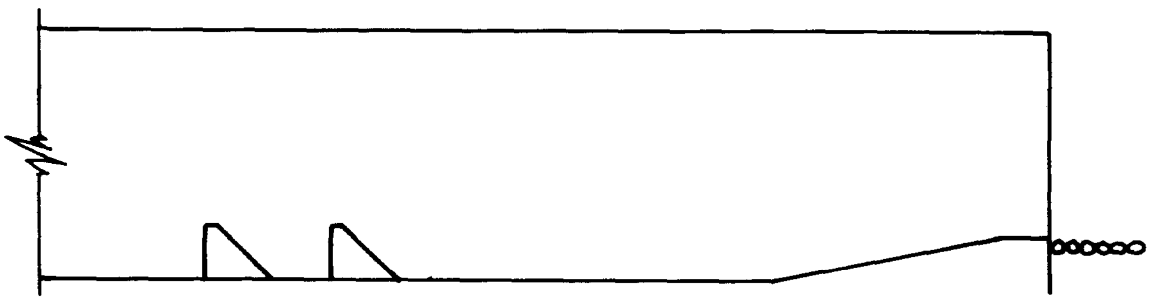
These basins designed for a single gate half or fully opened require higher baffle blocks than hydraulic-jump type basins. A second row of baffles is not required for maintaining the jump within the basin but is recommended to reduce attack on the downstream channel protection. These baffles should be 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. Reference item 2 of Appendix A for determining forces on baffle blocks. In cases where foundation requirements dictate a deep basin ($>d_2$), baffle blocks may not be required.

(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 paragraphs 5-9c(2) and 5-9c(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.

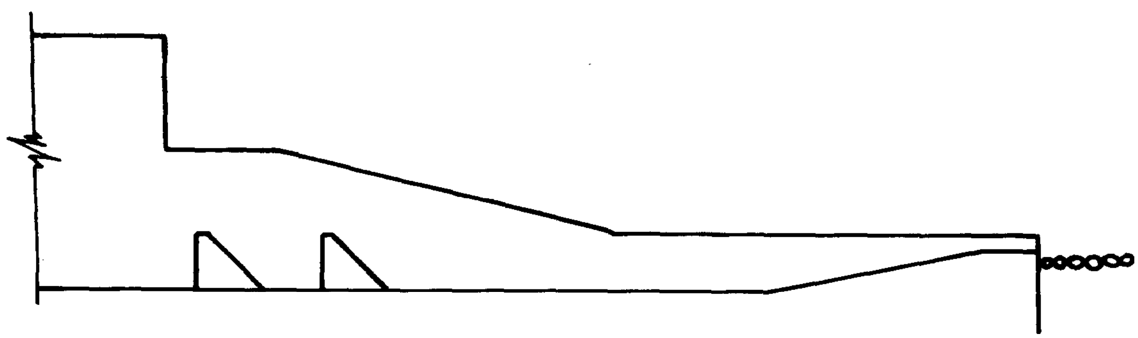
(6) End Sill. An end sill slope of 1V on 5H was effective in spreading the flow for single gate operation. The higher the end sill, the more effective it will be in spreading the jet during single gate operation, but there are limitations. The higher end sill results in shallower depths in the exit channel 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 below, should not exceed 0.86 for single gate guidance given in paragraph 5-9c. 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 percent 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 indicated 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 items 10, and 13-16 of Appendix A, the end-sill height varied 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.

(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 5-16a. This, too, is not a strict requirement. The Red River design is shown in Figure 5-16b. Adjacent project features and topography have a significant impact on training wall design. Reference EM 1110-2-1603 for determining hydraulic forces (static and dynamic) on stilling basin training walls.



a. CONVENTIONAL TRAINING WALL



b. RED RIVER # 3 TRAINING WALL

Figure 5-16. Training walls

(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.

(9) 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. Reference EM 1110-2-1602 for additional guidance relative to cavitation.

5-10. Approach Area.

a. Configuration. The approach to the spillway should be greater than three feet below the crest of the spillway. An approach depth of five feet is recommended because most discharge calibration data were taken with this depth. Approaches with depths less than three feet 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 tranch 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 feet and then sloped to the streambed at a rate not to exceed 1V on 20H.

b. 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.

$$\frac{D_{\%}}{\text{depth}} = C\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{1/2} \frac{V}{\sqrt{g\ \text{depth}}}\right]^{2.5} \qquad (5\text{-}13)$$

The following coefficients are recommended for riprap design in low turbulence open channel flow:

$D_{\%}$	Safe Design, C	Gradation	Thickness
D_{50} (Min)	0.44	Table 5-2	1.0 D_{100}(MAX)
D_{50} (Min)	0.30	Table 5-3	1.5 D_{100}(MAX)
D_{30} (Min)	0.375	d_{85}/d_{15} = 1.35-4.6	1.0 D_{100}(MAX)

The safe design C is equal to 1.25 times the C determined for incipient failure. See item 11 for additional information. Placement underwater requires an increase in thickness of 50 percent. Single gate operation will generally be the most severe with respect to design of upstream riprap but hinged pool operation (as described in paragraph 7-3(c)) 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.

5-11. Exit Area.

a. 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 above field and model experiences the following guides for preliminary layout are suggested. Begin the riprap with the top of the blanket 1 to 2 feet 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 feet from the face of the spillway training wall. These guides are illustrated in Plates 5-4 to 5-6 (example at end of this chapter).

b. 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. At the end sill the average velocity over the end sill can be 75 to 90 percent of the velocity without spreading. Results from items 10 and 13-16 of Appendix A show a wide variation in required riprap size. Use of 80 percent of the velocity over the riprap without spreading in the relation

$$V = 1.12 \left[2g \left(\frac{\gamma_s - \gamma_w}{\gamma_w} \right) \right]^{1/2} D_{50}(MIN)^{1/2} \tag{5-14}$$

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 Table 5-1 and Equation 5-14 is given in the following:

Project Name	Basin No.	Velocity over Riprap Without Spreading, ft/sec	D_{50} Model, feet	D_{50} Computed, feet
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
RR 3	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

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 Table 5-3 for placement in the dry. Thickness for placement in the dry should be $1.5D_{100}$(MAX) or $2.0D_{50}$(MAX), whichever is greater. Thickness for placement underwater should be increased 50 percent. The top of the riprap should be placed one to two feet 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 items 10 and 13-16 of Appendix A. 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 follows:

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

TABLE 5-2

Gradations for Riprap Placement in the Dry, Low Turbulence Zones

Percent Lighter by Weight	Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds	
Specific Weight = 155 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	81	32	159	63	274	110	435	174
50	24	16	47	32	81	55	129	87
15	12	5	23	10	41	17	64	27
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	649	260	924	370	1,268	507	1,688	675
50	192	130	274	185	376	254	500	338
15	96	41	137	58	188	79	250	105
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	2,191	877	3,480	1,392	5,194	2,078	7,396	2,958
50	649	438	1,031	696	1,539	1,039	2,191	1,479
15	325	137	516	217	769	325	1,096	462

(Continued)

TABLE 5-2 (Concluded)

Percent Lighter by Weight	Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds	
Specific Weight = 165 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	86	35	169	67	292	117	463	185
50	26	17	50	34	86	58	137	93
15	13	5	25	11	43	18	69	29
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	691	276	984	394	1,350	540	1,797	719
50	205	138	292	197	400	270	532	359
15	102	43	146	62	200	84	266	112
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	2,331	933	3,704	1,482	5,529	2,212	7,873	3,149
50	691	467	1,098	741	1,638	1,106	2,335	1,575
15	346	146	549	232	819	346	1,168	492
Specific Weight = 175 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	92	37	179	72	309	124	491	196
50	27	18	53	36	92	62	146	98
15	14	5	27	11	46	19	73	31
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	733	293	1,044	417	1,432	573	1,906	762
50	217	147	309	209	424	286	565	381
15	109	46	155	65	212	89	282	119
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	2,474	990	3,929	1,571	5,864	2,346	8,350	3,340
50	733	495	1,164	786	1,738	1,173	2,474	1,670
15	367	155	582	246	869	367	1,237	522

TABLE 5-3

Gradations for Riprap Placement in the Dry, High Turbulence Zones

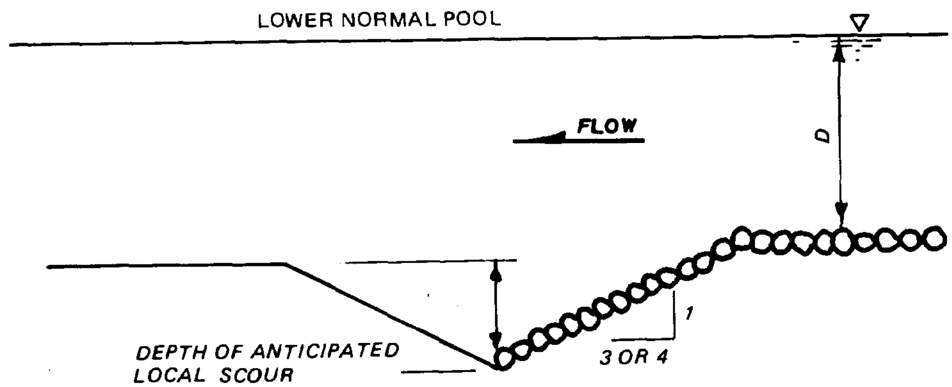
Percent Lighter by Weight	Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds	
Specific Weight = 155 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	24	10	47	19	81	32	129	52
50	7	5	14	9	24	16	38	26
15	4	2	7	3	12	5	19	8
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	192	77	274	110	376	150	500	200
50	57	38	81	55	111	75	148	100
15	28	12	41	17	56	23	74	31
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	649	260	1,031	412	1,539	616	2,191	877
50	192	130	305	206	456	308	649	438
15	96	41	153	64	228	96	325	137
Thickness =	60 Inches		66 Inches		72 Inches		78 Inches	
100	3,006	1,202	4,001	1,600	5,194	2,078	6,604	2,642
50	890	601	1,185	800	1,539	1,039	1,957	1,321
15	445	188	593	250	770	325	978	413
Thickness =	84 Inches		90 Inches		96 Inches		102 Inches	
100	8,248	3,299	10,145	4,058	12,312	4,925	14,768	5,907
50	2,444	1,650	3,006	2,029	3,648	2,462	4,376	2,954
15	1,222	516	1,503	634	1,824	770	2,188	923
Specific Weight = 165 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	26	10	50	20	86	35	137	55
50	11	5	21	10	36	17	58	27
15	5	2	11	3	18	5	29	9
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	205	82	292	117	400	160	532	213
50	86	41	123	58	169	80	225	106
15	43	13	62	18	84	25	112	33

(Continued)

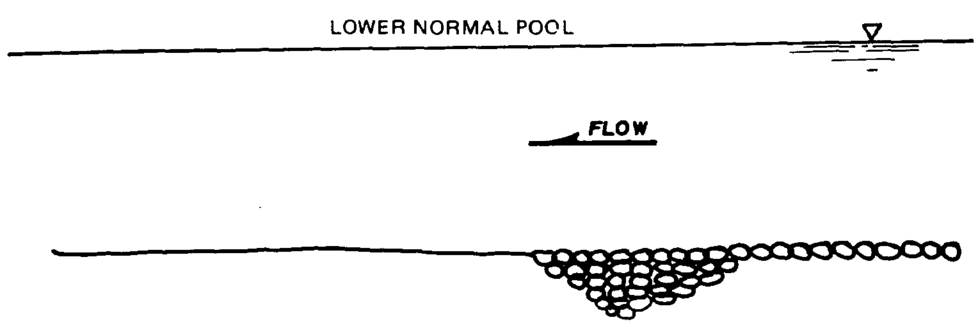
TABLE 5-3 (Concluded)

Percent Lighter by Weight	Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds	
Specific Weight = 165 lb/cu ft (continued)								
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	691	276	1,098	439	1,638	655	2,333	933
50	292	138	463	220	691	328	984	467
15	146	43	232	69	346	102	492	146
Thickness =	60 Inches		66 Inches		72 Inches		78 Inches	
100	3,200	1,280	4,259	1,704	5,529	2,212	7,030	2,812
50	948	640	1,262	852	1,638	1,106	2,083	1,406
15	474	200	631	266	819	346	1,041	439
Thickness =	84 Inches		90 Inches		96 Inches		102 Inches	
100	8,780	3,512	10,799	4,320	13,106	5,243	15,720	6,288
50	2,602	1,756	3,200	2,160	3,883	2,621	4,658	3,144
15	1,301	549	1,600	675	1,942	819	2,329	983
Specific Weight = 175 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	27	11	53	21	92	37	146	58
50	11	5	22	11	39	18	61	29
15	6	2	11	3	19	6	31	9
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	217	87	309	124	424	170	536	226
50	92	43	130	62	179	85	238	113
15	46	14	65	19	89	27	119	35
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	733	293	1,164	466	1,738	695	2,474	990
50	309	147	491	233	733	348	1,044	495
15	155	46	246	73	367	109	522	155
Thickness =	60 Inches		66 Inches		72 Inches		78 Inches	
100	3,394	1,357	4,517	1,807	5,864	2,346	7,456	2,982
50	1,006	679	1,338	903	1,738	1,173	2,204	1,491
15	503	212	669	282	869	367	1,105	466
Thickness =	84 Inches		90 Inches		96 Inches		102 Inches	
100	9,312	3,725	11,454	4,581	13,901	5,560	16,673	6,669
50	2,759	1,862	3,394	2,291	4,119	2,780	4,940	3,335
15	1,380	582	1,697	716	2,059	869	2,470	1,042

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 5-17. 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, EM 1110-2-1601 provides guidance for designing riprap end protection. 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 grout-filled bags have been used. Side-slope riprap is normally the same size as the invert. If required, riprap downstream of the $10d_2$ limit should be designed according to EM 1110-2-1601. Granular filters are recommended for riprap placement adjacent to structures. EM 1110-2-1901 presents guidance for filter design.



a. METHOD A - EXTEND TO ANTICIPATED SCOUR DEPTH



b. METHOD B - PROVIDE ROCK-FILLED TRENCH

Figure 5-17. Methods for transitioning from riprap to the unprotected downstream channel

5-12. 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, etc. The current most commonly used and recommended control is the tainter gate.

5-13. 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.

a. 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 feet, with lengths up to 125 feet on pile foundations and 150 feet on rock foundations.

b. 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 while the weight of a roller gate is about equally divided between the chain and the pier. Counterweights will reduce power required, but will add to the total weight of the structure. Tainter gates built to heights of 75 feet and lengths of 110 feet 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. Item 3 of Appendix A 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.

c. 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 feet. 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.

d. 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 Boule'dam (see Figure 5-18) are movable dams that are no longer being constructed but are still being used.

e. 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. The following advantages may be ascribed to tainter gate installations:

(1) Lighter lifting weight with smaller hoist requirements.

(2) Adaptable 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:

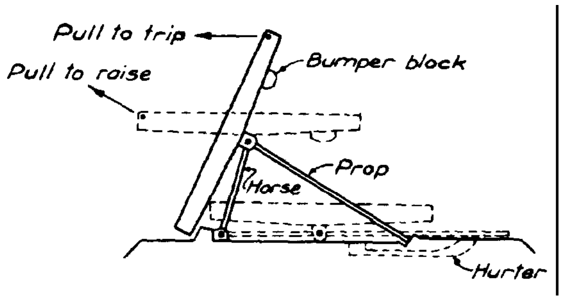
(1) Encroachment of radial arm on the water passage.

(2) The necessity for excessively long radial arms where the flood level, to be cleared, is extremely high.

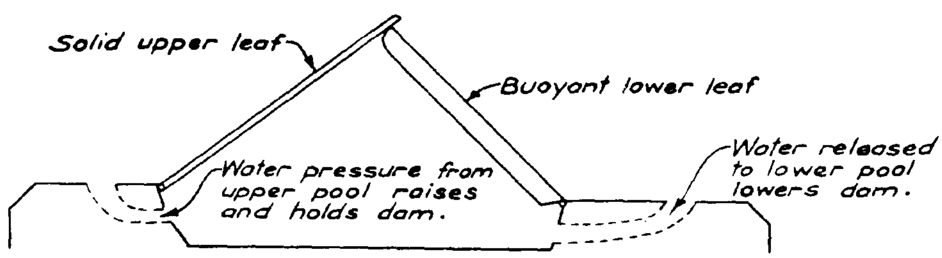
The advantages of a vertical-lift gate installation are:

(1) Provision of a clear gate opening with no encroachment, when raised, of any part of the gate structure on the water passage.

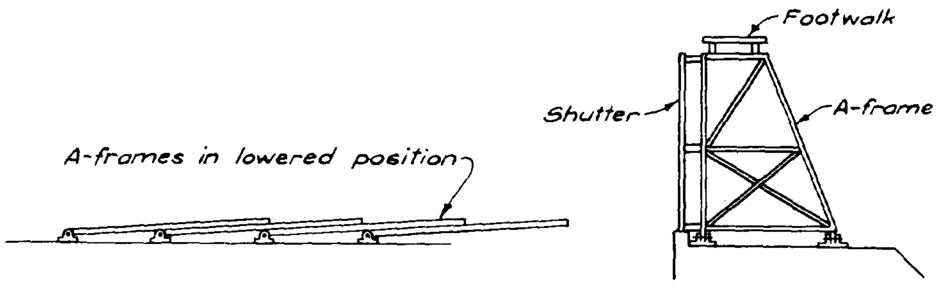
(2) More adaptable to extreme pool fluctuations in that it is lifted bodily out of the water.



WOODEN CHANOINE WICKET

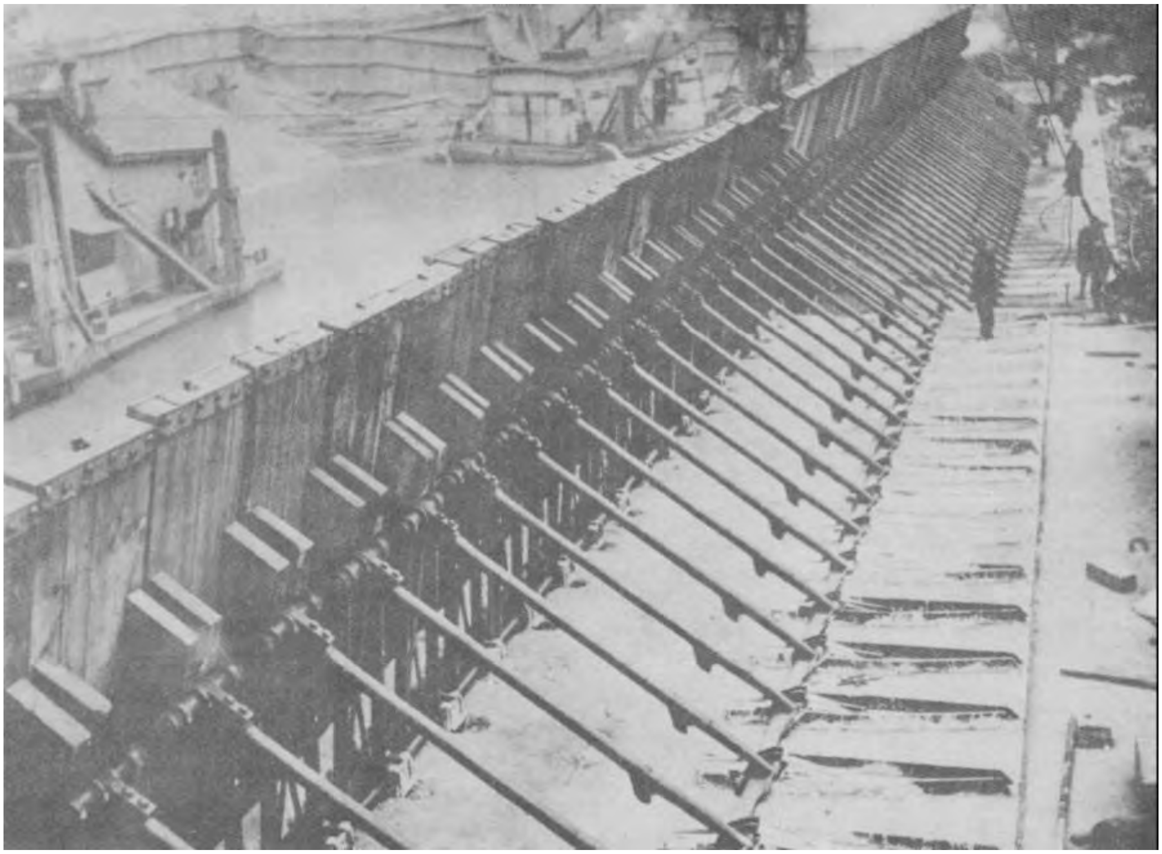


BEAR TRAP DAM

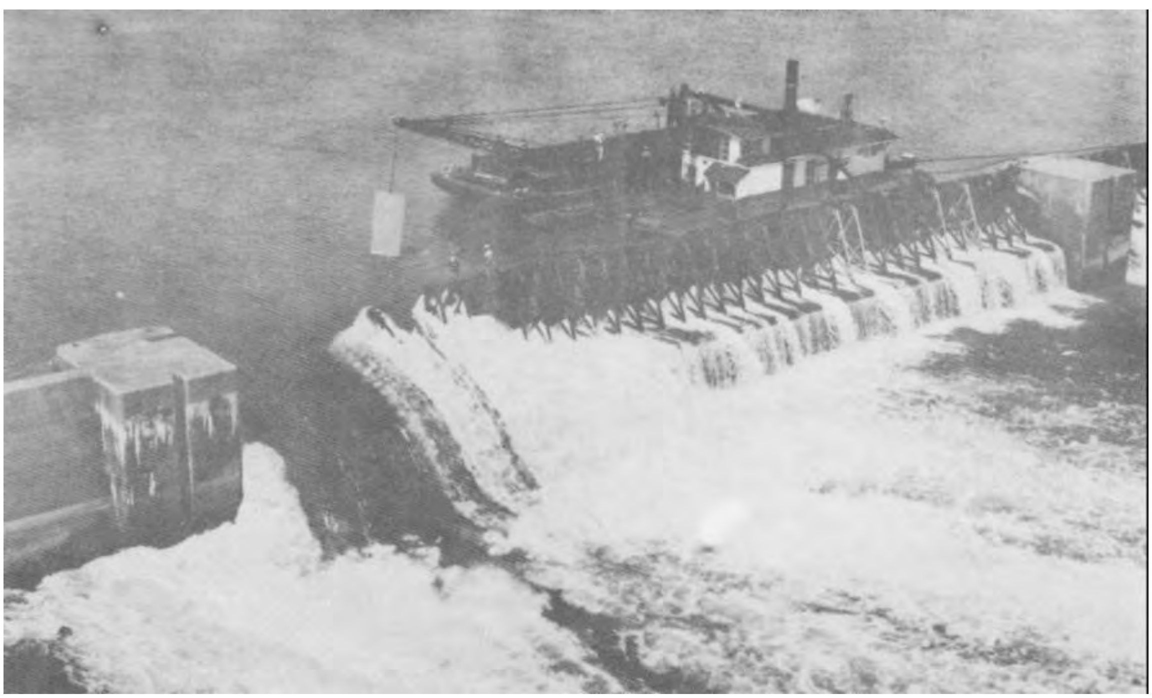


BOULE' DAM

Figure 5-18. Typical movable dams (Sheet 1 of 2)



Chanoine Wicket



Boule Dam
Figure 5-18. (Sheet 2 of 2)

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 adaption 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.

5-14. Tainter Gate Design. Reference is made to EM 1110-2-2702 and EM 1110-2-1603 for design guidance for tainter gates. Additional design guidance is given in the following paragraphs.

a. Gate Seal Design and Vibration. Many laboratory and field studies have been concerned with instabilities (gate vibration and oscillation) at CE projects. Reports given in items 4, 7, 8, 17, 19-21, 23, and 24 of Appendix A are representative of problems encountered and their solution. 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 breakoff point as possible. Supporting structural members downstream from the lip should be kept as high and narrow as possible. The Type C gate lip design (Figure 5-19), as used on Arkansas River Locks and Dams 8, 9, 13, and 14 gates, adequately meets these criteria. Severe vibrations adequate to eventually destroy the gates were experienced with Types A and B (see item 21, Appendix A).

(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 attached rigidly to the back side of the gate lip, as in type D design (Figure 5-19), is recommended. 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. The Type D seal designs are too inflexible to prevent leakage under these conditions. The Type A designs are very flexible but also vibration prone. Figure 5-20 shows an untested lip design developed to prevent this leakage problem. The notch in the gate sill may be subject to cavitation damage and should be tested under proposed operation conditions before being adopted.

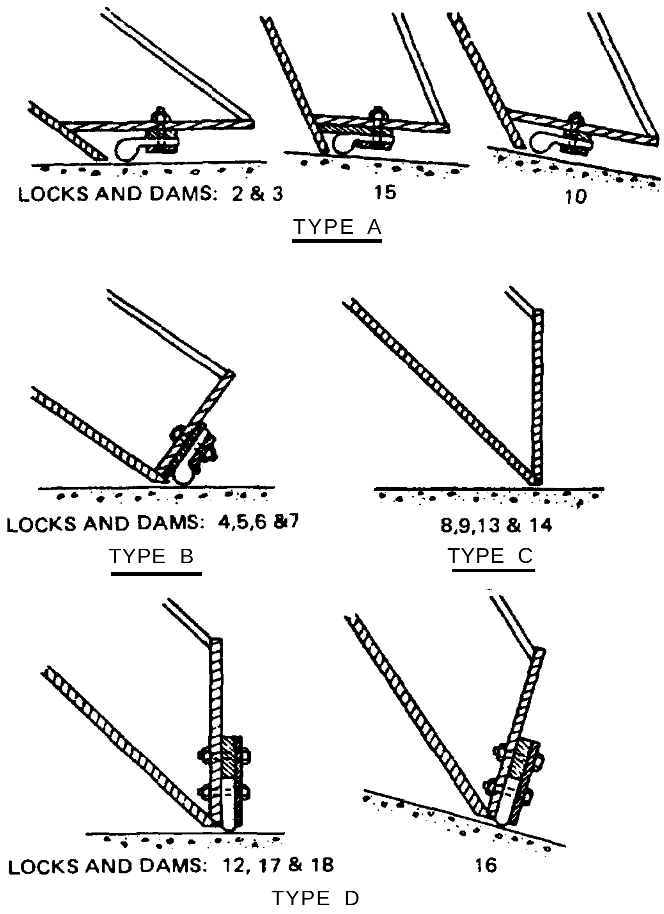


Figure 5-19. Gate lip design

(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. The normal J-bulb gate side seal is shown in Figure 5-21. Also shown is a modified rubber seal shape that was designed to maintain a seal over wide gap variations between the gate and the pier. This design should be tested on a prototype gate before extensive use on proposed projects.

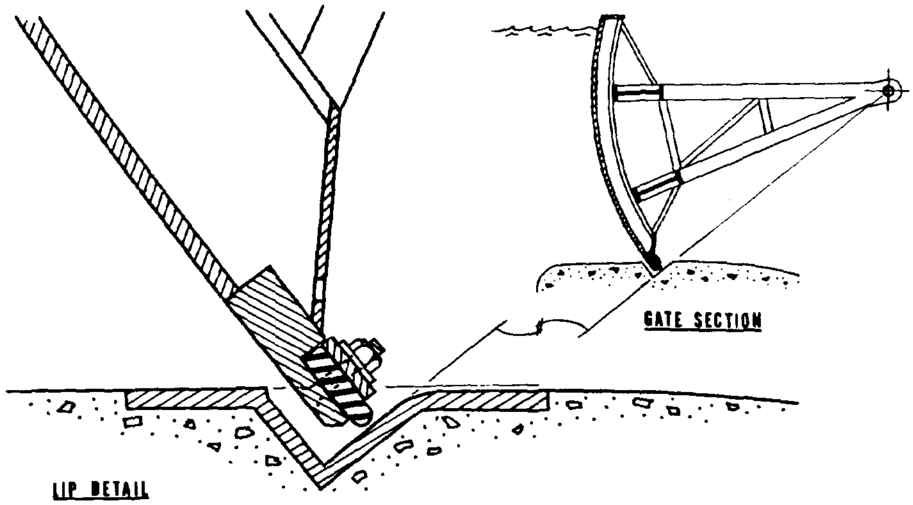


Figure 5-20. Bottom seal design for tainter gates, design proposed for vibration-free, leakage-free operation

(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.

b. 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.

c. 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.

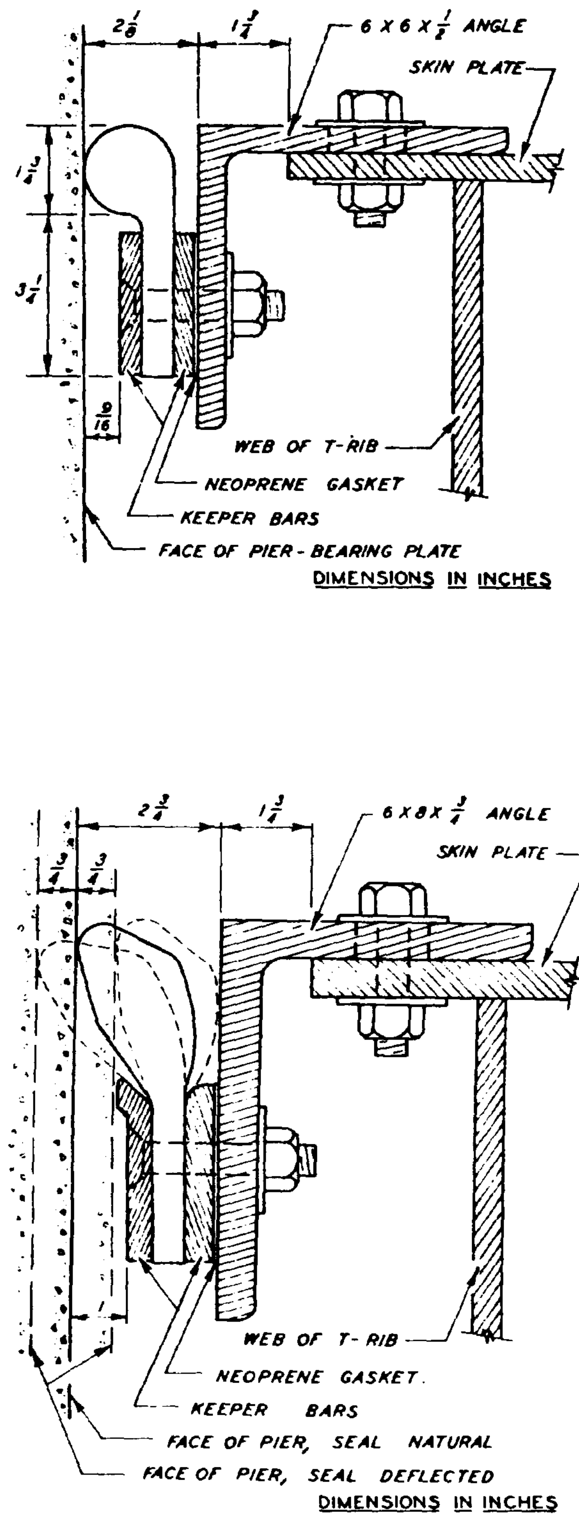


Figure 5-21. Gate side seals

d. Tainter Gate Trunnion Elevation. Trunnion elevation is set above most floods. Typical submergence allowed is a maximum of five to ten percent of the time.

e. 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.

f. 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.

g. 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.

h. Submergible Tainter Gates. Submergible tainter gates were developed to allow passage of ice without having to use large gate openings. Case histories of various types of submergible gates are presented in item 30 of Appendix A. 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 5-3. 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 Table 5-4 and a definition sketch is shown in Figure 5-22. 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.

5-15. Vertical-Lift Gate Design. Reference is made to EM 1110-2-2701 and EM 1110-2-1603 for design of vertical-lift gates.

5-16. Spillway Piers. The hydraulic performance and discharge capacity of spillways are affected by the pier designs. The following factors need to be considered.

a. 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 of item 10 and 13-16 projects, Appendix A, vary from 8 to 15 feet.

TABLE 5-4

Projects with Submergible Tainter Gates*

Lock and Dam (COE District)	River	H_D	Sub_G	C_G	D_G	Problem	Remarks
Greenup (Huntington)	Ohio	32.0	7.0	28.0	25.0	Yes	Problem: stilling basin and sill erosion and vibration Solution: submerged operation eliminated, plans and specs for modification as of Dec 1978
Meldahl (Huntington)	Ohio	30.0	7.0	28.0	23.0	Yes	Problem: stilling basin and sill erosion and vibration Solution: submerged operation eliminated
Markland (Louisville)	Ohio	34.0	7.0	33.0	27.0	Yes	Problem: vibration and jet through stilling basin and across end sill Solution: gate stops added and spillway curve modified. Submerged operation eliminated
McAlpine (Louisville)	Ohio	37.0	7.0	12.0	30.0	Yes	Problem: bed-material abrasion of sill Solution: submerged operation eliminated, spillway curve modified
Cheatham (Nashville)	Cumberland	26.0	7.0	19.0	19.0	Yes	Problem: vibration Solution: submerged operation eliminated, design modification being considered
New Cumberland (Pittsburgh)	Ohio	22.6	7.0	12.5	15.6	Yes	Problem: vibration, cavitation between gate and sill, and recreational craft hazard Solution: submerged operation eliminated
Pike Island (Pittsburgh)	Ohio	21.0	7.0	20.0	14.0	Yes	Problem: excessive leakage Solution: submerged operation eliminated
L&D No. 4 (Pittsburgh)	Monongahela	16.6	7.0	12.5	9.6	No	Movable crest or piggyback gate
Maxwell (Pittsburgh)	Monongahela	19.5	7.0	19.0	12.5	No	Movable crest or piggyback gate
L&D No. 11 (Rock Island)	Mississippi	11.0	8.0	12.0	3.0	No	13 gates
L&D No. 12 (Rock Island)	Mississippi	9.0	8.0	12.0	1.0	No	7 gates
L&D No. 13 (Rock Island)	Mississippi	11.0	8.0	12.0	3.0	No	10 gates
L&D No. 16 (Rock Island)	Mississippi	9.0	8.0	12.0	1.0	No	3 of 15 gates
L&D No. 17 (Rock Island)	Mississippi	8.0	8.0	8.0	0.0	No	8 gates
L&D No. 18 (Rock Island)	Mississippi	9.8	5.0	15.0	4.8	No	14 gates
L&D No. 20 (Rock Island)	Mississippi	10.0	3.0	17.0	7.0	No	6 of 40 gates
L&D No. 21 (Rock Island)	Mississippi	10.5	8.0	12.0	2.5	No	10 gates
L&D No. 22 (Rock Island)	Mississippi	10.5	8.0	17.0	2.5	No	1 of 10 gates

(Continued)

TABLE 5-4 (Concluded)

Lock and Dam (COE District)	River	H_D	Sub_G	C_G	D_G	Prob-lem	Remarks
St. Louis L&D No. 24	Mississippi	15.0	8.0	17.0	7.0	Yes	15-80' TG's, vibration, stress on trunion; submerged operation eliminated
St. Louis L&D No. 25	Mississippi	15.0	7.0	18.0	9.0	Yes	14-60' TG's, vibration stress on trunion; submerged operation eliminated
St. Louis L&D No. 26	Mississippi	24.0	3.0	27.0	21.0	No	30-40 TG's

* See Figure 5-22 for definition sketch.

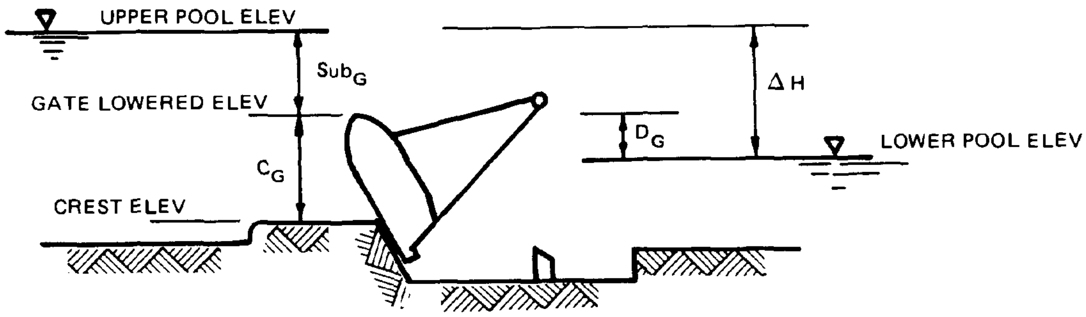


Figure 5-22. Definition sketch for variables used in Table 5-4

b. 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.

c. Pier Nose Shape. A semicircular pier nose shape is the most common and generally satisfactory design. An ogival shape (Type 3, HDC 111-5) was found to be only slightly more efficient than the semicircular shape (see item 6, Appendix A). All the Arkansas River navigation spillways have a curved nose leading to a 90-degree point (similar to ogival). 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.

d. 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 end of the piers. By recessing the posts back from the pier face, they will cause minimal flow disturbances.

5-17. 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, etc., would be hindered by large abutments. Abutment radius used on projects in items 10 and 13-16 of Appendix A were the same as the interior piers that equaled one-half of the pier width.

Section II. Design of Other Appurtenances

5-18. 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, etc. Model studies have shown that a navigable pass should have a minimum cross-sectional area 2-1/2 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 feet on the Ouachita to 932 and 1,248 feet 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 5-18) 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 (Figure 7-3).

5-19. 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 multi-level releases should be designed according to EM 1110-2-1602.

5-20. Fish Passage Facilities. Most fish passage facilities are located on rivers in the North Pacific Division (NPD). Engineers in NPD should be contacted for design information.

5-21. 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 5-23). Provisions to move ice past or through dams have

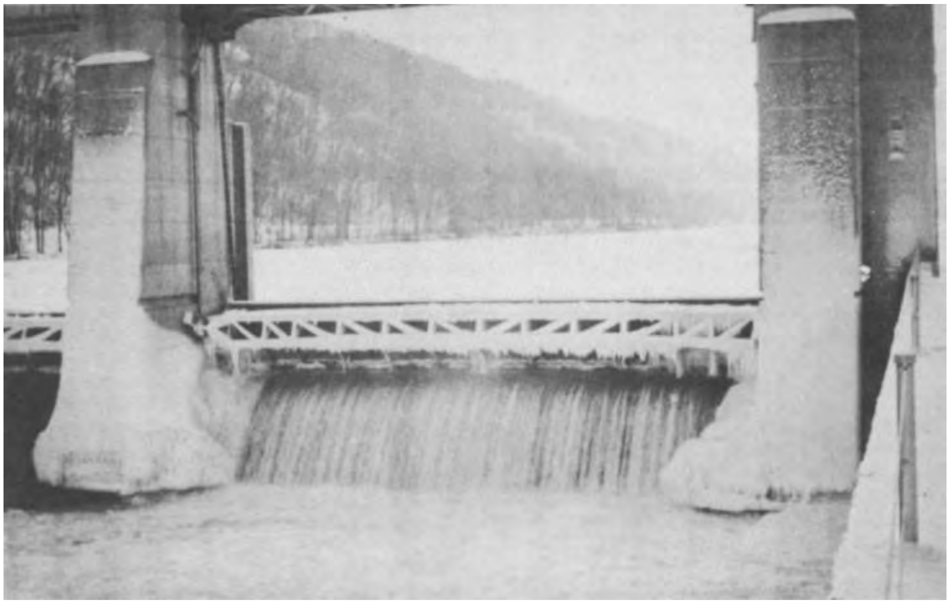


Figure 5-23. Ice on control gate

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, while 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. Physical models of ice control methods for specific projects can be made in the Ice Engineering Laboratory at the Corps of Engineers Cold Region Research and Engineering Laboratory in Hanover, N. H. EM 1110-2-1612 provides additional information on ice control methods.

Section III. Model Studies

5-22. General.

a. In the design of navigation dam spillways for major structures, a combination of analytical, laboratory, and field studies is usually needed. The laboratory studies can be physical or numerical models of flow conditions which are usually conducted at WES or ice studies for dams in cold regions which can be modeled at the Ice Engineering Laboratory at CRREL. Numerous problems in the design of spillways cannot always be solved satisfactorily without the use of model studies. Experience has shown that a model often can indicate more economical treatment of certain features which may reduce construction costs by many times the cost of the model. A model may reveal inadequacies in the basic design that would limit discharge capacity, result in costly maintenance, or even cause hazardous operation. It may be desirable to use hydraulic models for a specific project or for a typical case of a number of small structures. By using model studies, alternate plans and modifications can be tested within a relatively short time with all flow conditions that can be expected. Also, the design and operating engineers can observe conditions resulting with a particular arrangement and satisfy themselves as to the adequacy of the plan in addition to the advantages given above.

b. Examples of previous hydraulic models at WES used to solve spillway design problems are numerous. Among the most common usages 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 is normally used when approach conditions, flow over the spillway, and exit channel hydraulics are to be studied. A section model 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 the model limits since a two-dimensional model may not introduce flow patterns that can be addressed in a three-dimensional model.

c. 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.

d. 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.

e. A typical example of model study benefits is found in item 13 of Appendix A, where tests of a spillway as originally designed indicated that several modifications could improve performance and reduce project cost. Stilling basin tests demonstrated that the apron could be raised two feet to el 87.0 and still maintain an adequate jump under the most critical operating condition of one gate fully opened with the normal pool and minimum tailwater elevation expected. Two rows of baffles, eight feet high, seven feet wide, and eight feet apart, were found to be more beneficial than the original single row in dissipating energy and maintaining the hydraulic jump. Pier extensions 37 feet long and 23 feet high were essential for the elimination of return flows and eddies experienced during single-gate operations. A lower terminal apron elevation and riprap on a 1V-on-20H upslope were required downstream of the stilling basin to prevent the formation of a secondary jump over the horizontal downstream riprap protection. Multiple- or single-gate openings greater than six feet created a secondary jump with the original design basin and low tailwaters. The recommended design stilling basin eliminated the secondary jump and provided satisfactory energy dissipation for both normal and emergency operating conditions. Other changes from the original design included eliminating the approach trench upstream of the spillway, eliminating the go-degree curved endwall downstream of the left stilling basin training wall, and shortening the right training wall between the gated and ungated spillways from 115 to 40 feet. The approach trench was removed to prevent irregular flow conditions. The go-degree curved endwall tended to magnify wave action on the left bank. Reducing the length of the right training wall was economically beneficial since any length beyond 40 feet did not improve hydraulic performance. A considerable reduction in the excavation requirements along the right downstream bank was recommended to improve flow patterns and decrease construction costs. This recommended reduction in width decreased eddy action, eliminated much of the return flow along the right bank, and produced better flow patterns for both single- and multiple-gate operations.

Section IV. Example Design

5-23. Known Information. From optimization study (see Appendix D for example), a six-gated structure is required having the following dimensions:

Normal Upper Pool Elevation = 140

Normal Lower Pool Elevation = 110

Crest Elevation = 100

Maximum High Water Elevation = 165

Tailwater Stage Exceeded 10 Percent of the Time = 139

Tailwater Buildup Is Slow

Channel Invert Elevation = 100

Left Side of Spillway Adjacent to Lock Wall

Right Side of Spillway Has 1V-on-3H Side Slope

Use Standard, Nonsubmergible Tainter Gate

Gate Width = 60 feet = (Width of Monolith - Pier Width)

Pier Width = 10 feet

Unit Weight of Available Stone = 165 lb/ft^3

Riprap to be Placed in the Dry

5-24. Development of Design.

a. Upstream Face and Radius - Use vertical upstream face with a five-foot radius (due to 40-foot head) connecting the upstream face and horizontal crest.

b. Structural requirements usually dictate length of horizontal crest from upstream face to beginning of downstream face. Past projects have used approximately 110 percent of the head on the crest. Distance = 1.10(40) = 44 feet.

c. Downstream Face:

H = Normal Pool - Crest Elevation = 40 feet

V_o (for parabolic drop) = $\sqrt{2g(1/3)H}$ = 29.3 ft/sec

$$X^2 = \frac{2V_o^2 Y}{g} = \frac{2(29.3)^2 Y}{32.2} = 53.3Y \qquad \text{(5-1 bis)}$$

This is the steepest slope recommended for a head of 40 feet; use X^2 q 55Y. The downstream face shaped according to this equation will not experience severely negative pressures and the jet will adhere to the downstream face of the crest. Point at which slope equals 1V on 1H:

$$Y = \frac{X^2}{55}$$

$$\text{Slope} = \frac{dY}{dX} = \frac{2x}{55}$$

$$\text{For slope} = 1 = \frac{2x}{55}\ ,\ X = 27.5\ ,\ Y = 13.75$$

d. Discharge Rating - Free uncontrolled flow is needed for input into stilling basin design. Some of the other three flow regimes require the stilling basin apron elevation and will not be computed in this step.

$$Q = C_F L H^{3/2} \qquad \text{(5-2 bis)}$$

Using Figure 5-9, and using an abutment contraction coefficient since the adjacent bays are not operating, the following table results for discharge through a single bay.

Upper Pool Elevation	H_e/R*	$K_a/2$**	$L_{effective}$, feet	H/B_c	C	Q, cfs/bay
100	0	--	60.0	0	--	0
105	1	0.015	59.85	0.11	3.00	2,007
110	2	0.021	59.6	0.23	3.04	5,730
115	3	0.027	59.2	0.34	3.07	10,557
120	4	0.036	58.6	0.45	3.09	16,196
125	5	0.04	58.0	0.57	3.11	22,548
130	6	0.042	57.5	0.68	3.15	29,762
135	7	0.044	56.9	0.80	3.19	37,584
140	8	0.046	56.3	0.91	3.24	46,163

* R = l/2 pier width for use in HDC 111-3/1
** See paragraph 5-7c

e. Stilling Basin Apron Elevation - Use a single gate fully opened, normal upper pool, and minimum tailwater (which equals the normal lower pool since there is a slow tailwater buildup) to determine the apron elevation. The unit discharge into the basin is

$$q = \frac{Q}{W} = \frac{46163}{60} = 769.4 \text{ cfs/ft}$$

<u>Assume</u> Stilling basin apron elevation = 75

Solve Equation 5-8 by trial and error for V_1 and d_1 using no energy loss between upper pool and stilling basin apron

$$140 = 75 + \frac{V_1^2}{2g} + d_1$$

$$V_1 = \frac{q}{d_1} = \frac{769.4}{d_1}$$

we are actually solving

$$140 = 75 + \frac{\left(\frac{769.4}{d_1}\right)^2}{2g} + d_1$$

The solution is $d_1 = 13.35$ feet

and

$$V_1 = \frac{769.4}{13.35} = 57.6 \text{ ft/sec}$$

$$F_1 = \frac{V_1}{\sqrt{gd_1}} = 2.78$$

$$\frac{d_2}{d_1} = 0.5\left(\sqrt{1 + 8F_1^2} - 1\right) = 3.46$$

$$d_2 = 3.46(13.35) = 46.2 \text{ feet}$$

Check assumed stilling basin elevation using tailwater equal to 85% d_2

(Factor = 0.85 in Equation 5-11)

$$110 - 75 \neq 0.85(46.2)$$

$$35 \neq 39.3$$

A new stilling basin apron elevation must be assumed until the above equation is satisfied. The correct solution is an apron elevation = 69.0.

$$d_1 = 12.55 \text{ feet}$$

$$V_1 = \frac{769.4}{12.55} = 61.31 \text{ ft/sec}$$

$$F_1 = \frac{61.31}{[(32.2)(12.55)]^{1/2}} = 3.05$$

$$d_2 = 48.25 \text{ feet}$$

f. Basin Length - Distance from beginning of basin to 1V-on-5H upslope $L_2 = 2d_1F_1^{1.5} = 133.7$ ft.

g. Baffles - Height = $0.25d_2$ = 12.06, use 12 feet. Distance to first row = $1.3d_2$ = 62.7 feet. Distance between upstream faces of baffle = 2(12) = 24 feet.

h. Pier Extensions - Extend 57.7 feet into basin. Use five feet wide beyond main piers and use top elevation of 112 (two feet above lower normal pool).

i. End Sill - Use end-sill height = $0.15d_2$ = 7.2 feet, use 7.0.

j. Training Wall - Extend right training wall to end of basin at a top elevation of 112.

k. Approach Area Configuration - Use approach five feet below crest, horizontal for 50 feet, and slope up to streambed for 100 feet at 1V on 20H.

l. Approach Area Riprap - Average velocity = 769.4/(140 - 95) = 17.1 ft/sec. Using Equation 5-13, we have the following choices:

C	Thickness in D_{100}	Gradation Table	D_{50}(MIN), feet	W_{50}(MIN), lbs	Thickness inches
0.44	1.0 D_{100}(max)	5-2	1.4	258	30
0.30	1.5 D_{100}(max)	5-3	1.0	86	33

Gradations other than those given in Tables 5-2 and 5-3 could be used by determining D_{30} in Equation 5-13 with a blanket thickness of 1.0 D_{100}MAX).

m. Exit Channel Configuration - The top of the end sill will be at 69 + 7 = 76.0. Place top of riprap 1.0 foot below top of end sill. Slope exit channel up to streambed for 500 feet at 1V on 20H.

Exit Channel Riprap - Velocity over end sill w/o spreading = (769.G/llO - 76) = 22.6 ft/sec, use 0.80(22.6) = 18.1 ft/sec in Equation 5-14.

$$D_{50}(MIN) = 2.5 \text{ feet}$$

$$W_{50}(MIN) = 1,302 \text{ pounds}$$

Using gradation Table 5-3 for high turbulence, use thickness = 78 inches immediately below end sill.

Distance, feet	Thickness, inches
$3d_2 = 150$	78
$3d_2 = 150$	66
$2d_2 = 100$	48
$2d_2 = 100$	33

Adjacent to the lock wall, spreading of the single gate fully opened will be inhibited and rock size cannot be decreased as rapidly as given in the above table. Use 78-inch thickness for the first 300 feet then 66-inch thickness for the remaining 200 feet. Provide trench of riprap at downstream end to protect toe.

o. Tainter Gate Design - For this example design, a gate radius of 1.25 times the damming height of the gate will be used. In reality, this radius can depend on other factors not considered in this example. The trunnion elevation will be placed one foot above the stage that is exceeded 10 percent of the time.

$$R = 1.25(40) = 50 \text{ feet}$$

$$\text{Trunnion elevation} = 139 + 1 = 140 \text{ feet}$$

The gate seat location will be at the beginning of the parabolic drop.

p. Pier Design - Use semicircular pier noses located in the same plane as the upstream face of the structure.

q. Abutments - Abutment radius should be one-half the pier width or five feet.

r. Discharge Rating -

(1) Submerged Uncontrolled - Use the d'Aubuisson equation (5-5) with K = 0.90 since bay width = 60 ft. An iterative solution is required.

H, feet	h, feet	Approach Area, ft^2	K	AH, feet	Q, cfs All Gates
12.5	10	6,409	0.9	2.5	37,750
11.43	10	5,991	0.9	1.43	29,007
10.53	10	5,642	0.9	0.53	17,954
10.15	10	5,495	0.9	0.15	9,629
25.0	20	11,550	0.9	5.0	109,550
22.86	20	10,637	0.9	2.86	85,009
21.05	20	9,875	0.9	1.05	53,004
20.30	20	9,562	0.9	0.30	28,743
37.50	30	17,159	0.9	7.50	201,805
34.29	30	15,674	0.9	4.29	157,164
31.58	30	14,444	0.9	1.58	98,623
30.46	30	13,942	0.9	0.46	54,132

Results are plotted in Plate 5-1 along with the values for free uncontrolled flow.

(2) Free Controlled Flow - Using the coefficients presented in Figure 5-11:

H, feet	G_0, feet	C_g	Q, cfs/bay
30	1	1.0	2,636
30	6	0.69	10,912
30	14	0.58	21,401
20	1	0.90	1,937
20	6	0.65	8,393
10	1	0.82	1,248
10	6	0.54	4,930

Results are plotted in Plate 5-2 along with the curve for free uncontrolled flow. For heads greater than 30 feet or gate openings greater than 14 feet, HDC 320-4 to 320-7 must be used. The trunnion height above crest "a" equals 40 feet. This results in the ratio a/R = 40/50 = 0.8 which requires interpolation between HDC 320-5 and HDC 320-6. Determine L/P = 44/5 = 8.8 and find adjustment factor C2 = 1.03 .

H	G_o	G_o/R	H/R	C_1(a/R = 0.5)	C_1(a/R = 0.9)	C_1(a/R = 0.8)	Q, cfs/bay
30	20	0.40	0.60	0.495	0.528	0.520	28,230
40	20	0.40	0.80	0.517	0.555	0.546	34,230
40	14	0.28	0.80	0.545	0.605	0.590	25,138
40	6	0.12	0.80	0.622	0.723	0.698	13,130

(3) Submerged Controlled Flow - This type of flow requires a different rating curve for each gate opening. Using Figure 5-12 for c_{gs} , B = 31 feet:

G_o, ft	H, ft	h, ft	h/G_o	C_{gs}	ΔH, ft	Q, cfs/bay
1	30	10	10	0.076	20	1,636
1	25	10	10	0.076	15	1,416
1	20	10	10	0.076	10	1,156
1	15	10	10	0.076	5	818
1	30	20	20	0.037	10	1,126
1	25	20	20	0.037	5	796
6	30	10	1.67	0.47	20	10,114
6	25	10	1.67	0.47	15	8,759
6	20	10	1.67	0.47	10	7,152
6	15	10	1.67	0.47	5	5,057
6	30	20	3.33	0.23	10	7,000
6	25	20	3.33	0.23	5	4,950
14	30	20	1.43	0.58	10	17,652
14	25	20	1.43	0.58	5	12,482

Results are presented in Plate 5-3 along with the curves for free controlled flow.

s. Plan and profiles of the completed structure are given in Plates 5-4 to 5-6.

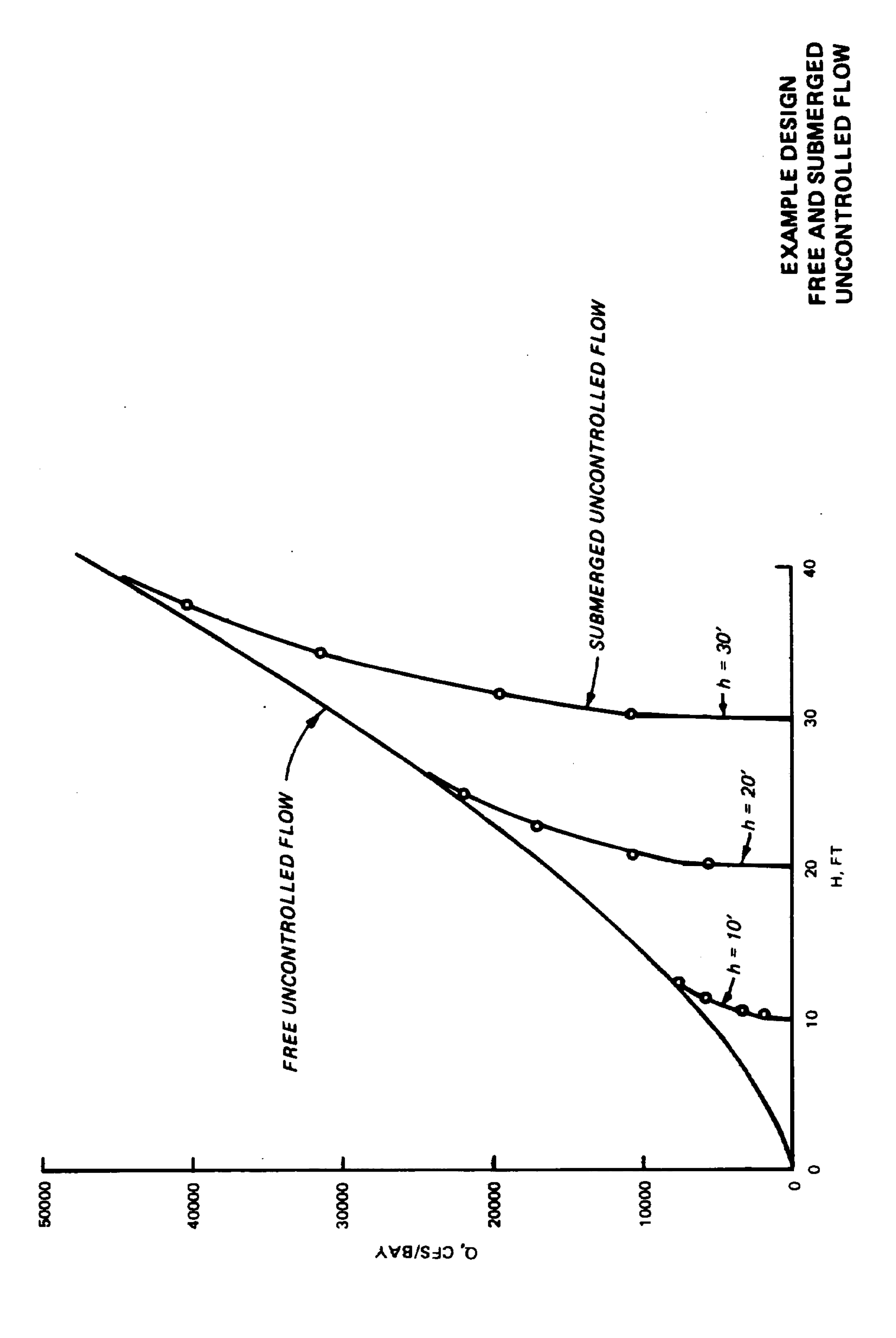
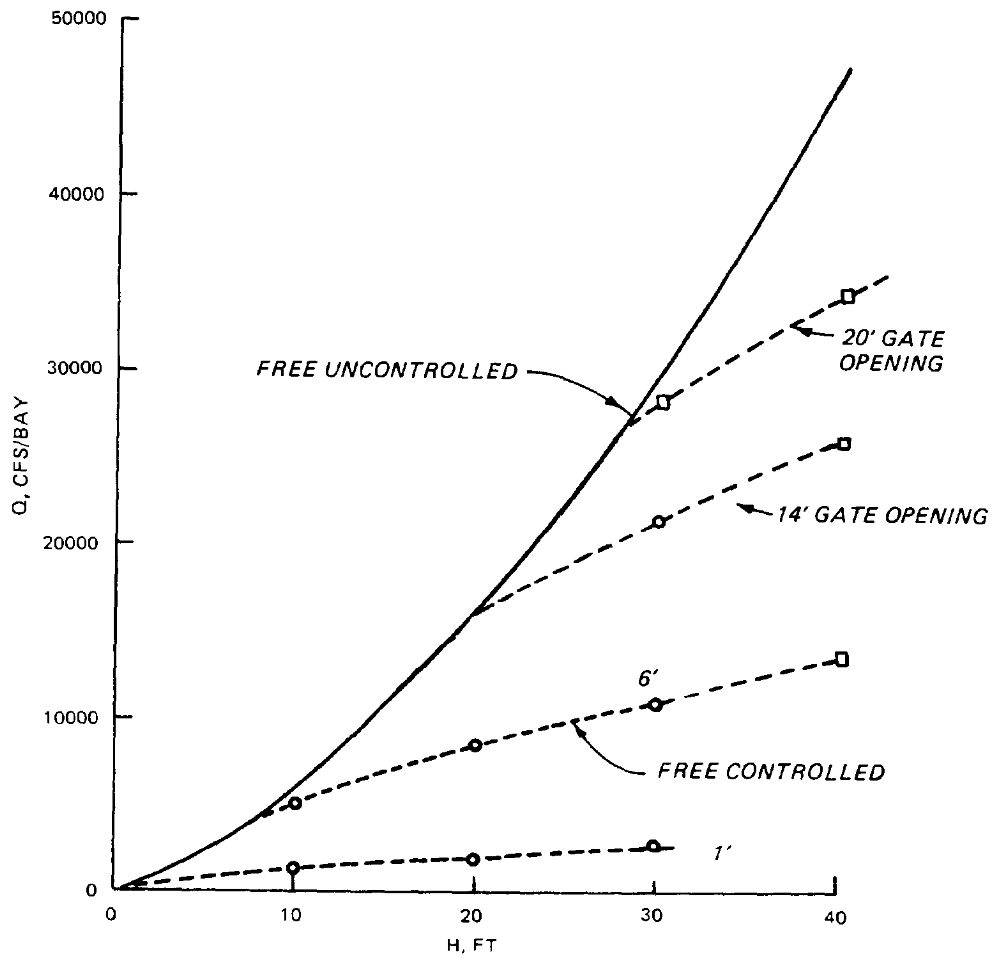

EXAMPLE DESIGN
FREE AND SUBMERGED
UNCONTROLLED FLOW
SUBMERGED UNCONTROLLED FLOW
FREE UNCONTROLLED FLOW
h = 30'
h = 20'
h = 10'
Q, CFS/BAY
H, FT
0
10000
20000
30000
40000
50000
0
10
20
30
40

PLATE 5-1

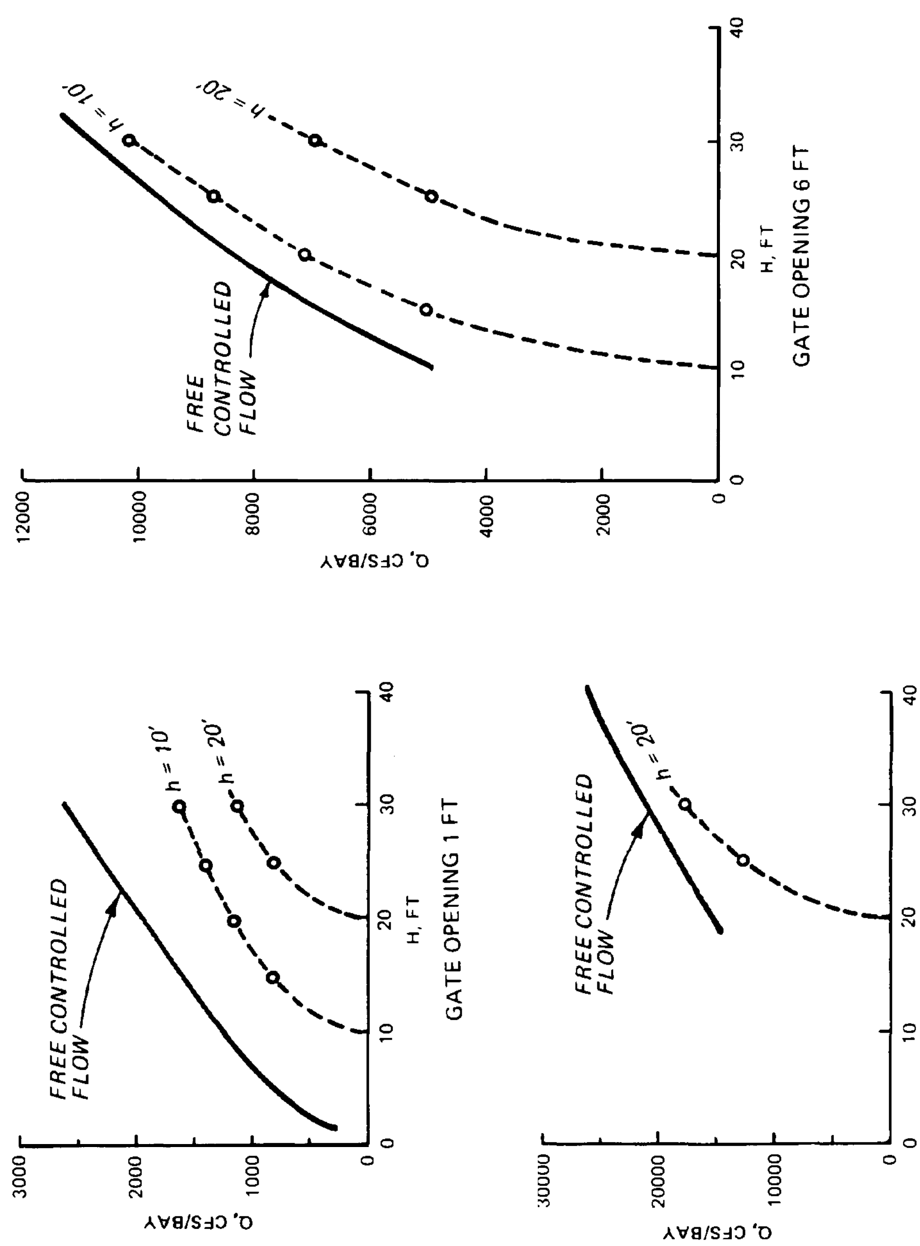


LEGEND

o FIGURE 5-11
□ HDC 320-4 THRU 320-7

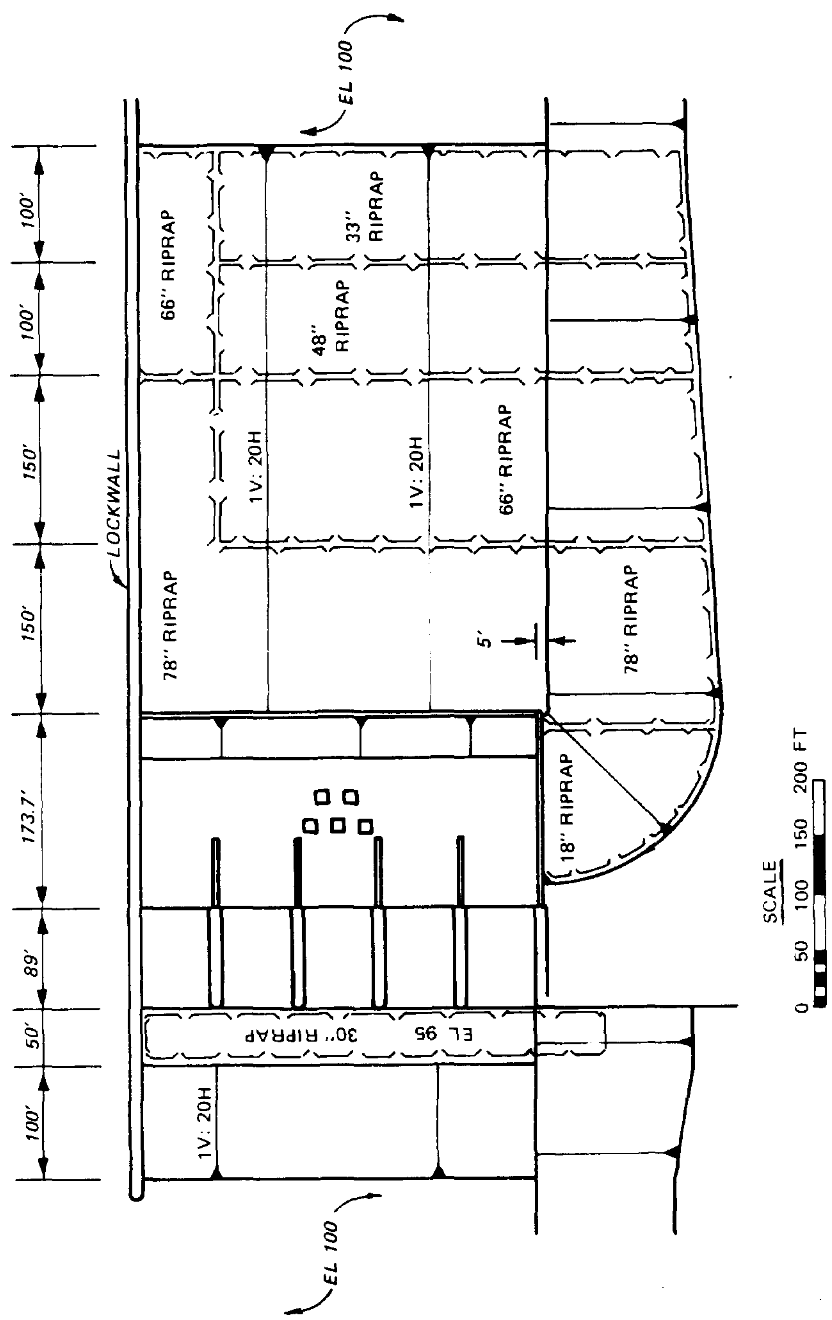
EXAMPLE DESIGN
FREE CONTROLLED AND
FREE UNCONTROLLED FLOW

PLATE 5-2



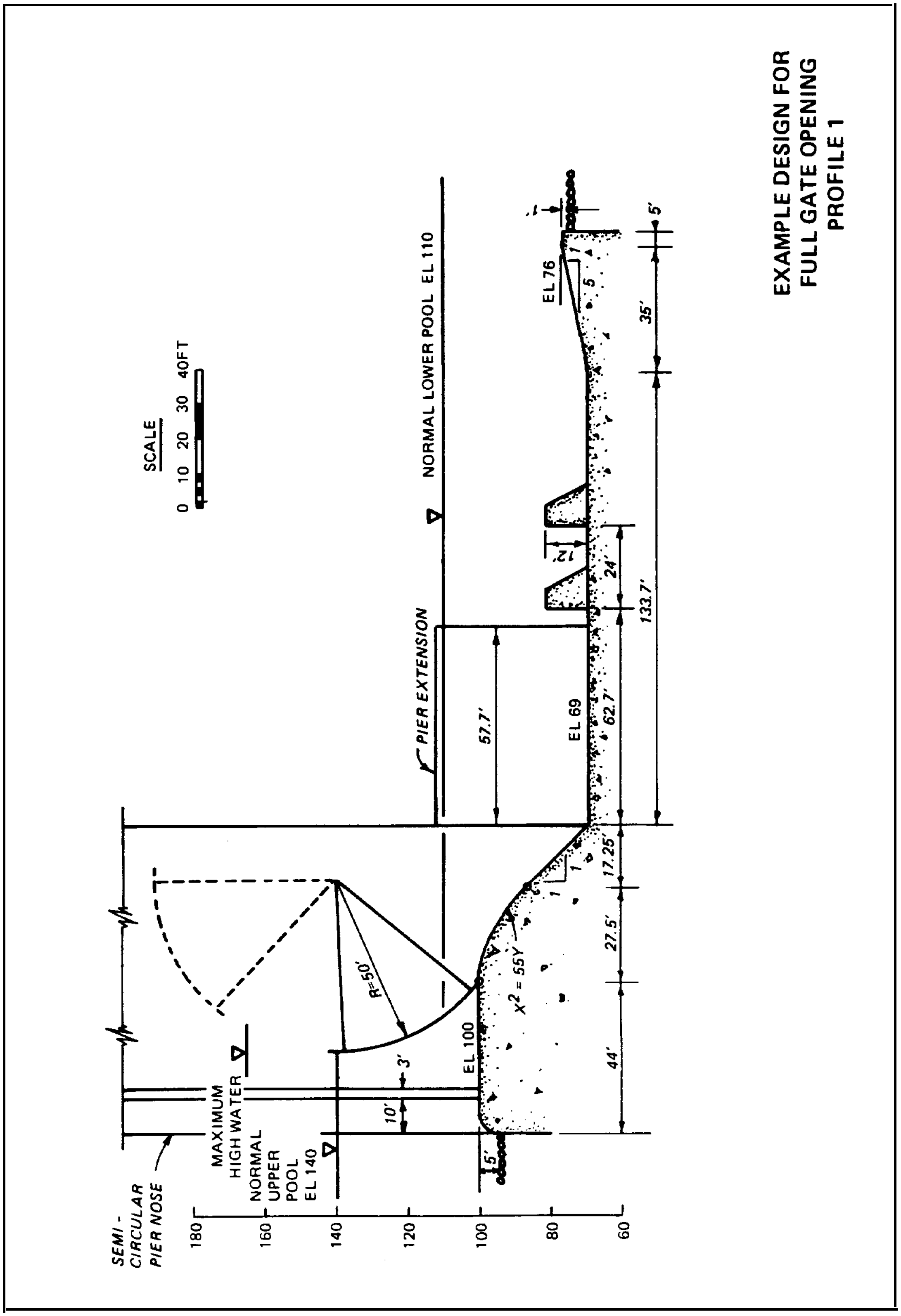
EXAMPLE DESIGN
FREE CONTROLLED AND SUBMERGED
CONTROLLED FLOW

PLATE 5-3



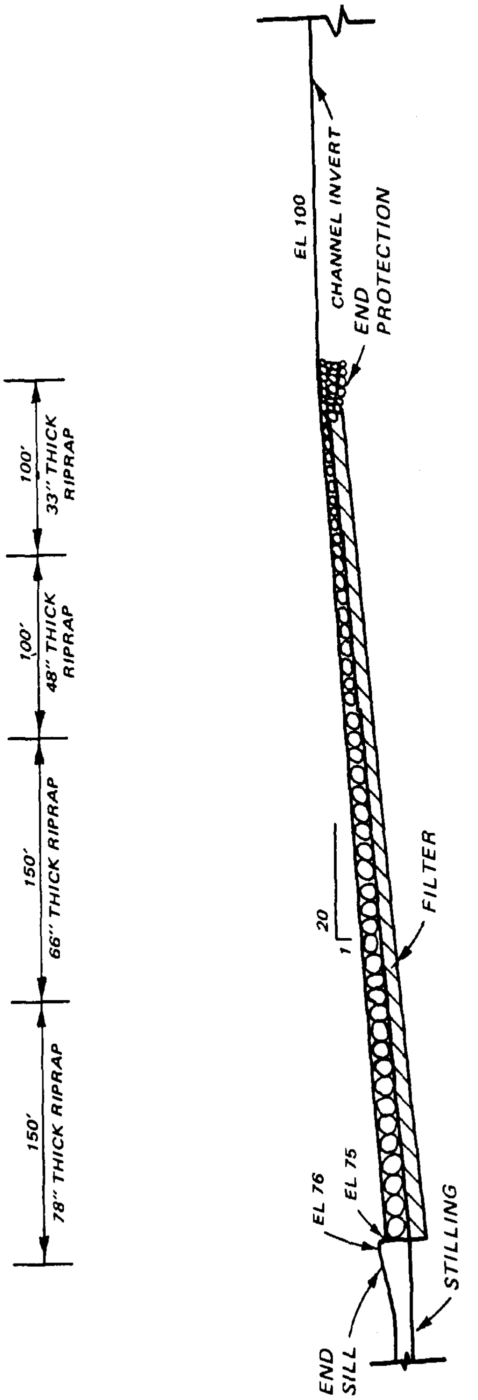
EXAMPLE DESIGN FOR
FULL GATE OPENING
PLAN VIEW

PLATE 5-4



EXAMPLE DESIGN FOR
FULL GATE OPENING
PROFILE 1

PLATE 5-5



**EXAMPLE DESIGN FOR
FULL GATE OPENING
PROFILE 2**

PLATE 5-6

CHAPTER 6

PROJECT CONSTRUCTION

Section I. General

6-1. 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 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.

a. Flooding. When designing a cofferdam scheme, an important design consideration is to limit upstream flooding to acceptable levels. Al though 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.

b. 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.

6-2. 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 to be feasible, since the inclusion even of a relatively small portion of the dam in the first stage 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.

6-3. 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 stages. In general, economy dictates as few construction stages as possible, because of the cost and time delay associated with removal and replacing of earth embankments or sheet piling for cofferdam cells. However, the number of stages 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. As an example, in an analysis performed by the Little Rock District for the proposed Dardanelle Lock and Dam project on the Arkansas River, it was determined that a four-stage diversion plan was the most economical (Figure 6-1). This plan required the construction of 62-foot-diameter cofferdam cells to a maximum height of 59 feet, requiring 7,400 tons of piling with a total estimated cost of $6 million. Another alternative was a three-stage plan with a stabilizing beam inside the cofferdam that required the construction of 52.5-foot-diameter cells to a maximum height of 66 feet above bedrock. This alternative required 10,200 tons of piling with a cost of $6.8 million. Thus the four-stage plan required less sheet piling because of a smaller increase in upstream stages and it was therefore recommended for construction. It also had the advantage of the reduced headwater flooding. Navigation structures can be constructed in a single phase cofferdam scheme, resulting in significant time and cost savings. Dam 2 Spillway on the Arkansas River is an example. The existing river was not disturbed; the spillway was located on the alignment of a proposed river channel cutoff; the spillway was constructed; and finally the river was diverted to flow through the completed structure. Once diverted, an additional phase was required to construct the closure structure across the old river channel. The time for raising of the pool and the rate of rise must be carefully chosen. From a project operation standpoint, it is preferable to raise the pool as soon as conditions permit; however, environmental, commercial, recreational, and social considerations must be taken into account also. In addition, adequate flow must be maintained during the pool rise to prevent degradation of river water quality. Generally, on rivers with existing open-river navigation, locks must be constructed while maintaining navigation at the same time. To supplement flow capacity lost during later construction phases, the completed lock can be used as a floodway to reduce the effect of induced flooding, but only after careful analysis of hydraulic and structural consequences of such action.

Section II. Cofferdams

6-4. General Schemes. Cofferdams are temporary structures in the river providing an enclosure to permit the construction of the entire or a part of the navigation dam. In the following, a few typical cofferdam layout schemes are presented as illustrations of possible solutions. However, this does not

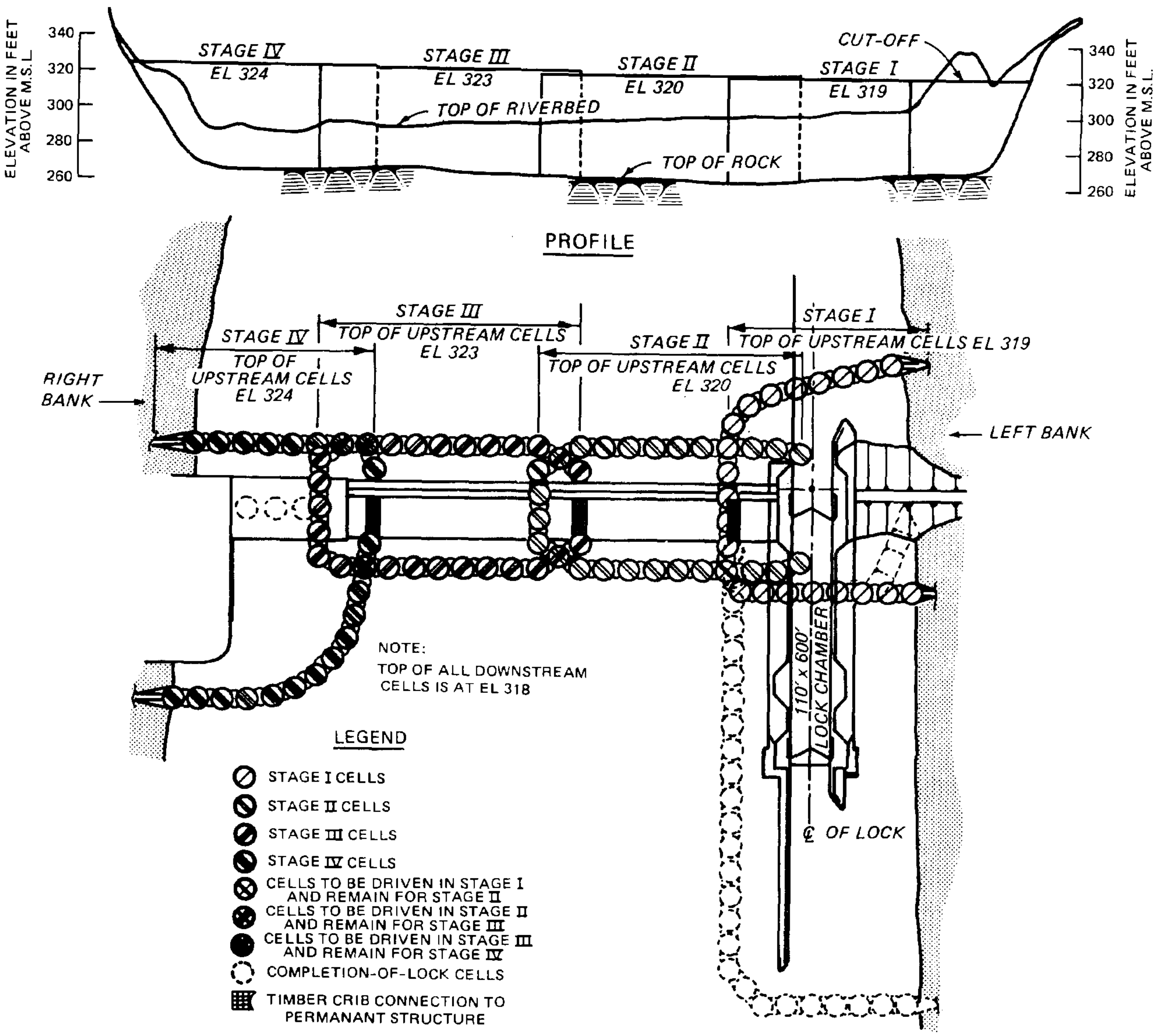


Figure 6-1. Four-stage diversion plan

imply that these are the only possible alternatives; the design should be tailored for specific local conditions. Of interest to the hydraulic engineer is the method of establishing the top elevation of the cofferdam based on the discharge and/or stage frequency-durationships of the river. This subject frequency relationships will be more fully discussed later in this chapter. A typical cofferdam layout for the construction of Greenup Lock and Dam on the Ohio River is shown in Figure 6-2. In this case, two- and three-stage cofferdam layouts were studied, and the three-stage layout was selected to avoid high currents adversely affecting navigation. Another possibility is shown in Figure 6-3 which indicates the construction plan for the replacement of Lock and Dam 26 on the Mississippi River. As shown, 6-l/2 gatebays were constructed during the first stage. River traffic used the opening between the first stage and the Illinois bank during this phase. The second stage involves the construction of the lock, and the remaining one-half gatebay, during which phase the river traffic uses the opening between the second stage cofferdam and the Illinois bank. Riverflows pass through the navigation opening between the second stage cofferdam and the Illinois bank and that portion of the spillway completed during the first stage. In the third stage, the remaining gatebays are constructed and the lock is available for river traffic. Another example of a typical cofferdam scheme is shown in Figure 6-4, which is the recommended layout for the Newburgh Lock and Dam project on the Ohio River. In this case, two alternatives were studied: a three-stage plan involving partial construction of the dam, and a two-stage plan which involves the construction of all 10 gatebays in a single cofferdam. It was found that the recommended two-stage construction was more economical, in terms of initial construction cost and resulted in a shorter construction period for the project. River traffic used the opening between the first stage cofferdam and the left riverbank during the first stage construction, and was directed to the locks upon completion of the first stage. In the second stage, the fixed-weir section of the project was constructed providing nine gatebays for flow passage.

6-5. 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 which 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, 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 following formula:

$$P = \frac{N!p^{i}(1 - p)^{N-i}}{i!(N - i)!}$$

Where P is the probability of obtaining, in N trials, exactly i events having a probability of p of occurring in a single trial. For the special case where $i = 0$, the formula becomes:

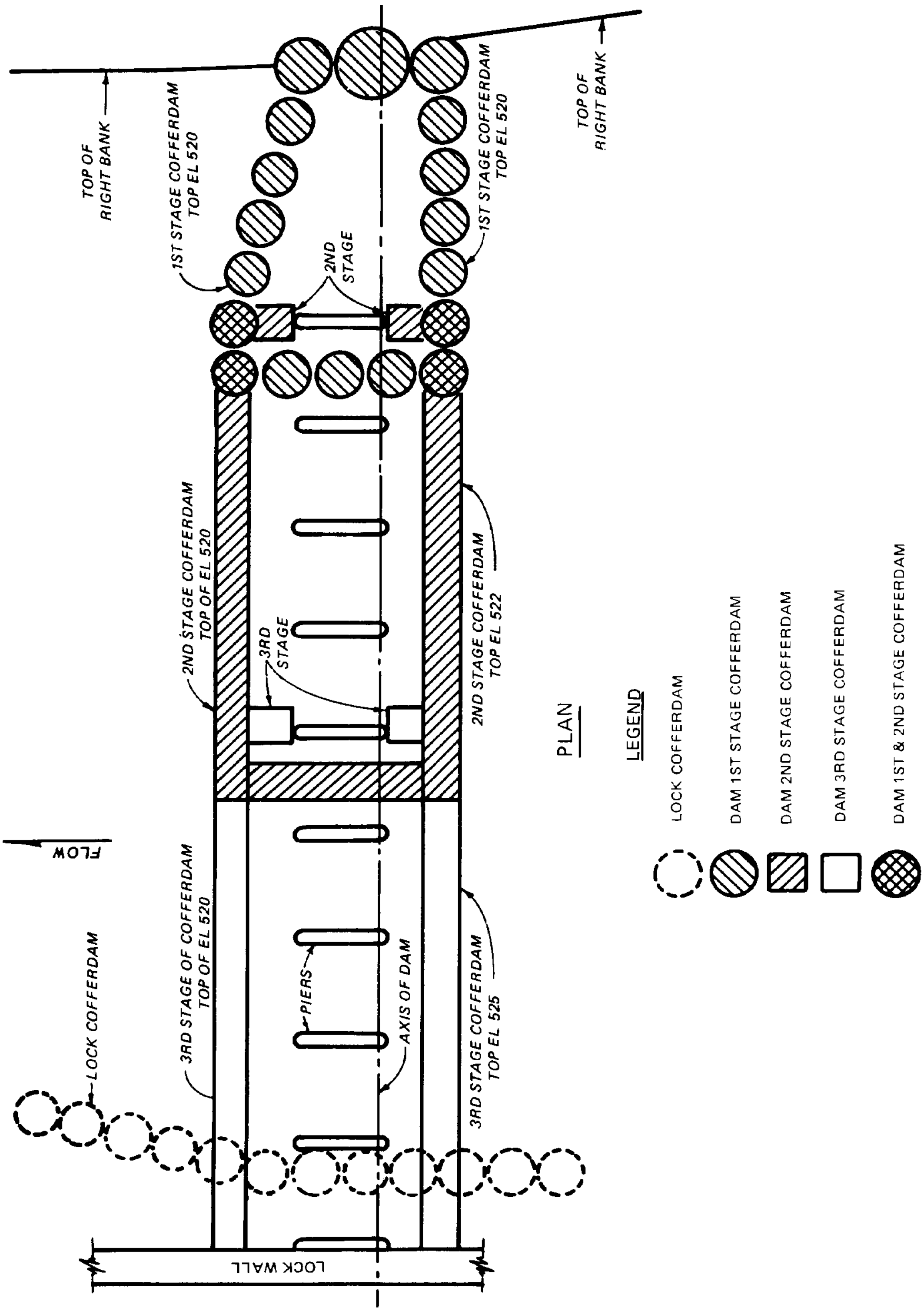


Figure 6-2. Cofferdam scheme, Greenup Lock and Dam, Ohio River

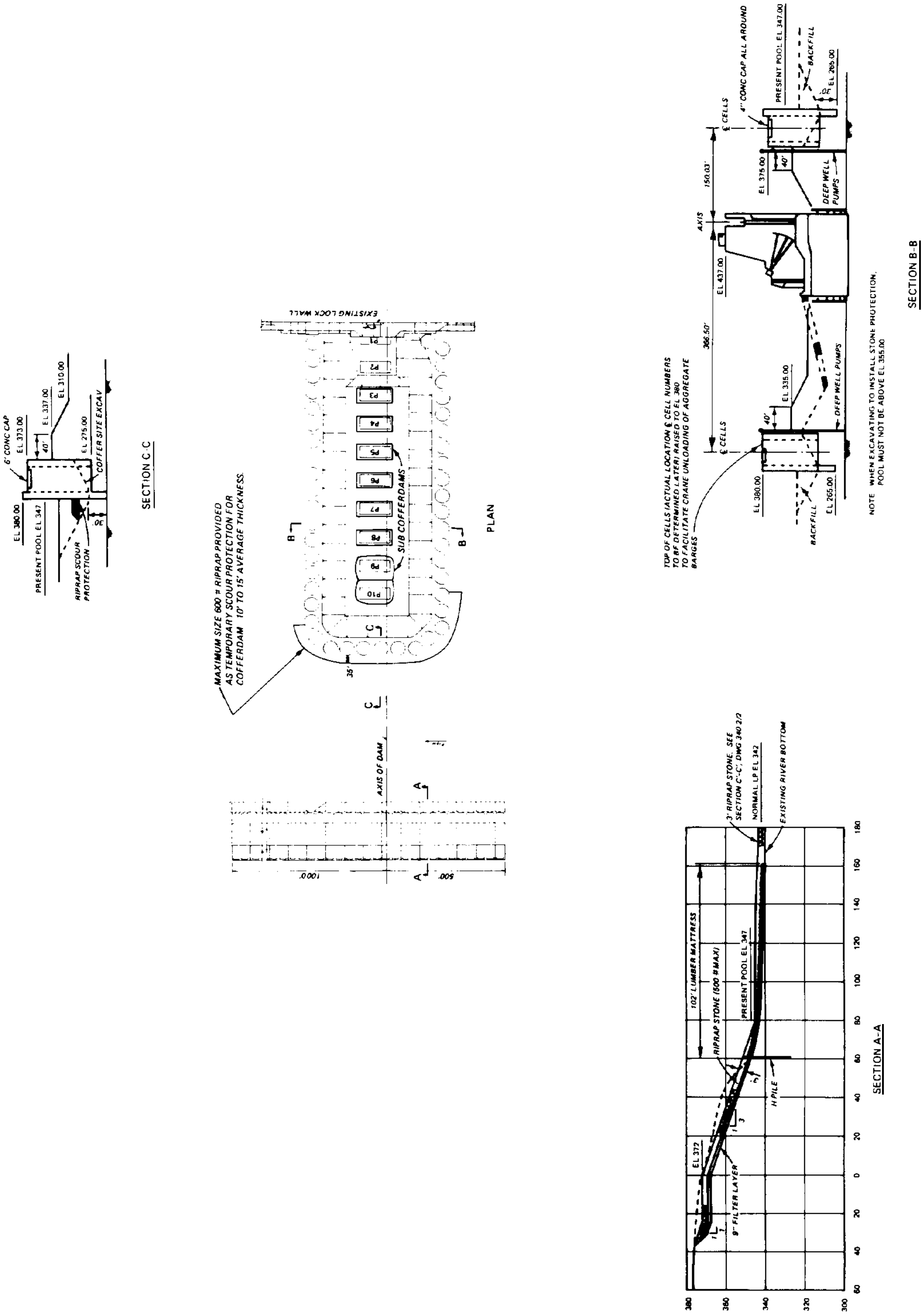


Figure 6-3. Cofferdam scheme, Newburgh Locks and Dam, Ohio River

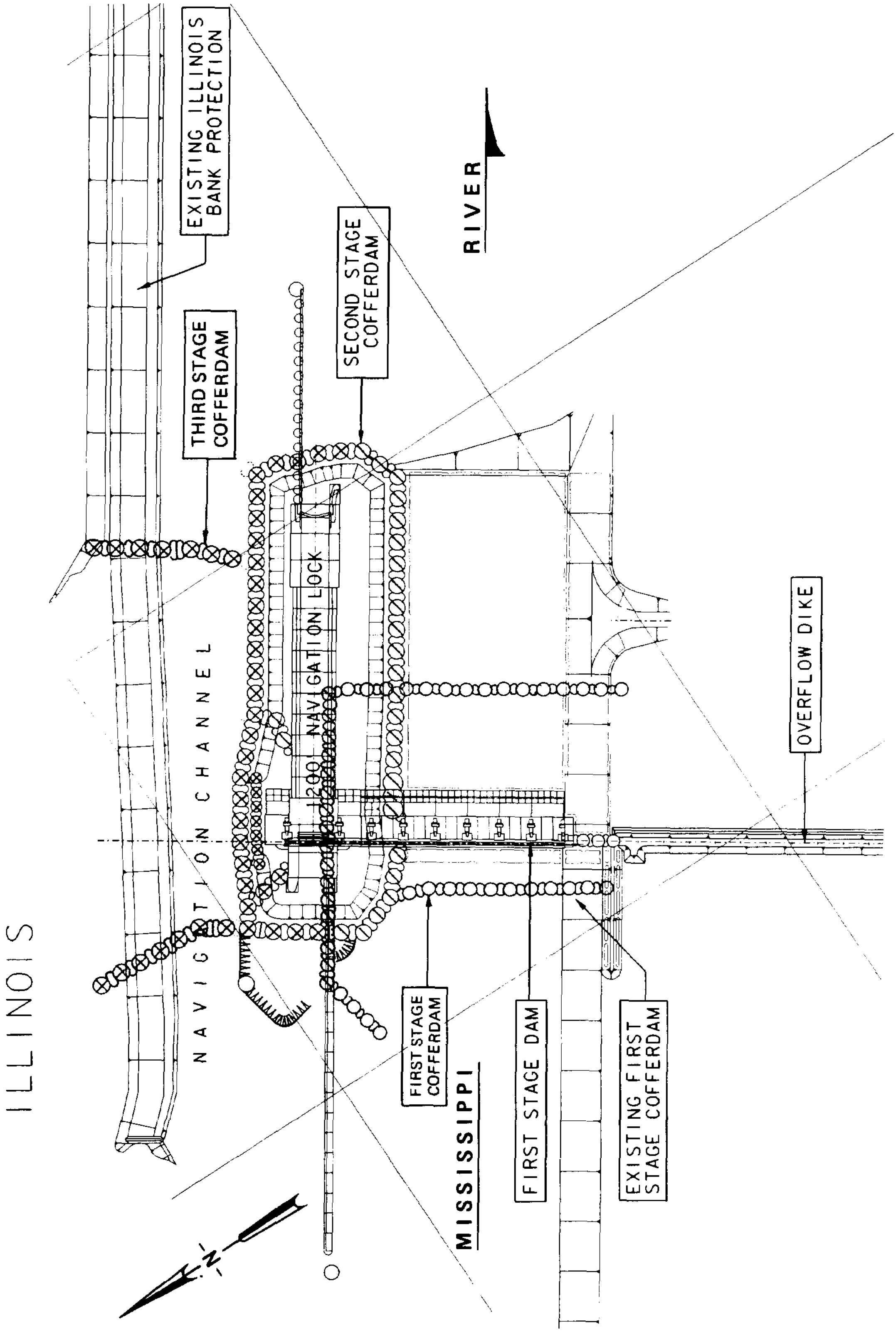


Figure 6-4. Cofferdam scheme, Lock and Dam 26, Mississippi River

$$P = (1 - p)^N$$

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:

$$P = 1 - (1 - p)^N$$

For example, in a project with a three-year construction period, N = 3 . To analyze the flooding for a lo-year flood, p = 0.1 . Therefore

$$P = 1 - (1 - 0.1)^3 = 0.271$$

or, a 27.1 percent chance that a lo-year flood will occur one or more times in a given three-year period. The total probable flooding cost for each height of cofferdam can be computed by the formula:

$$C_t = P[(D)(C_1) + C_2]$$

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

6-6. 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.

6-7. Example Determination of Cofferdam Heights. The following example is similar to a design of the cofferdam height at the Columbus Lock and Dam on the Tennessee-Tombigbee Waterway. The estimated flooding costs, the flood damage costs, the comparative cofferdam construction costs, the method of duration analysis, and the high discharge duration curve are shown in Figures 6-5 to 6-9, respectively. In Figure 6-10, the estimated probable

FIXED COST PER FLOODING

Downtime	-------------------	10 days @ $10,500/day	= $105,000
Pumping and Cleanup	-------------------	10 days @ $ 7,000/day	= $ 70,000
Damage Cost	-------------------	Lump sum	= $ 50,000
Investment Cost	-------------------	10 days @ $ 3,000/day	= $ 30,000
Liquidated Damages	-------------------	10 days @ $ 500/day	= $ 5,000
			$260,000

TOTAL COST PER FLOODING

$260,000 + [(D) x ($10,500 + $3,000 + $500)]

$260,000 + (D x $14,000)

where D = Duration of flood in days before pumping and cleanup can start

NOTES : Experience and professional judgment were used in estimating the cost for each of the items used in determining a realistic total cost for flooding of the cofferdam. The equipment downtime cost was based on the assumption that the cofferdam flooding would occur during peak concrete placement at which time the maximum amount of equipment would be on the job site. Pumping and cleanup cost was based on an average time of 10 days to pump out and clean up the protected area. This cost includes extra equipment for the pumping and cleanup crews. Damage cost was estimated considering equipment loss, duplication of work effort caused by berm and slope sloughing, wood form loss, and damage to prepared foundations. Investment cost is the estimated daily interest cost to the Federal Government during construction. Since the construction is on the critical path, downtime during the work phase will extend the total project completion time. This cost was derived by dividing the present estimated value for interest during construction by the construction period to get a one-day cost. The liquidated damages cost is the extra cost incurred by the Corps of Engineers for each day past the schedule completion date.

Figure 6-5. Estimated flooding costs

	FLOOD FREQUENCY, YEARS								
	2	4	5	6	8	10	25	50	100
Probability (3-yr Const)	0.704	0.578	0.488	0.421	0.330	0.271	0.115	0.059	0.030
Q (cfs)	74,000	83,000	90,500	98,000	110,000	118,000	160,000	200,000	240,000
Stage (Elev)	166.4	167.4	168.1	168.6	169.3	169.6	171.5	173.0	174.4
Cofferdam (Elev)	169.4	170.4	171.1	171.6	172.3	172.6	174.5	176.0	177.4
Duration B %	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00
Duration D %	0.29	0.20	0.14	0.11	0.07	0.05	—	—	—
Duration C (B − D) + 2 %	1.36	1.40	1.43	1.43	1.47	1.48	1.50	1.50	1.50
Duration A B − C %	1.64	1.60	1.57	1.55	1.53	1.52	1.50	1.50	1.50
Duration Days A × Const P	17.96	17.52	17.19	16.97	16.75	16.64	16.43	16.43	16.43
Duration Costs	251,440	245,280	240,660	237,580	234,500	232,960	230,020	230,020	230,020
Duration Cost × Prob	177,013	141,772	117,442	100,021	77,385	63,132	26,452	13,571	6,901
Fixed Costs × Prob	183,040	150,280	126,880	109,460	85,800	70,460	29,900	15,340	7,800
Flooding Costs	360,053	292,052	244,322	209,481	163,185	133,592	56,352	28,911	14,701

NOTE: Natural ground level is 155 where dike will be breached for flooding of the construction area.
Q = 29,400 cfs Duration = 3.00%
Downtime, investment loss, and liquidated damage cost per day = $14,000 (Duration Cost).
Fixed cost of flooding = $260,000.

Figure 6-6. Flood damage costs

TOP OF COFFERDAM ELEVATION, FEET		COMPACTED FILL	STRIPPING	TOTAL COST OF VARIABLES
UPSTREAM	DOWNSTREAM	$	$	$
169.5	168.5	406,100	15,400	421,500
171.5	170.5	510,500	17,200	527,700
173.5	172.5	626,500	19,000	645,500
175.5	174.5	754,400	20,900	775,300
177.5	176.5	893,800	22,700	916,500
179.5	178.5	1,047,200	24,500	1,071,700

Figure 6-7. Comparative cofferdam construction costs

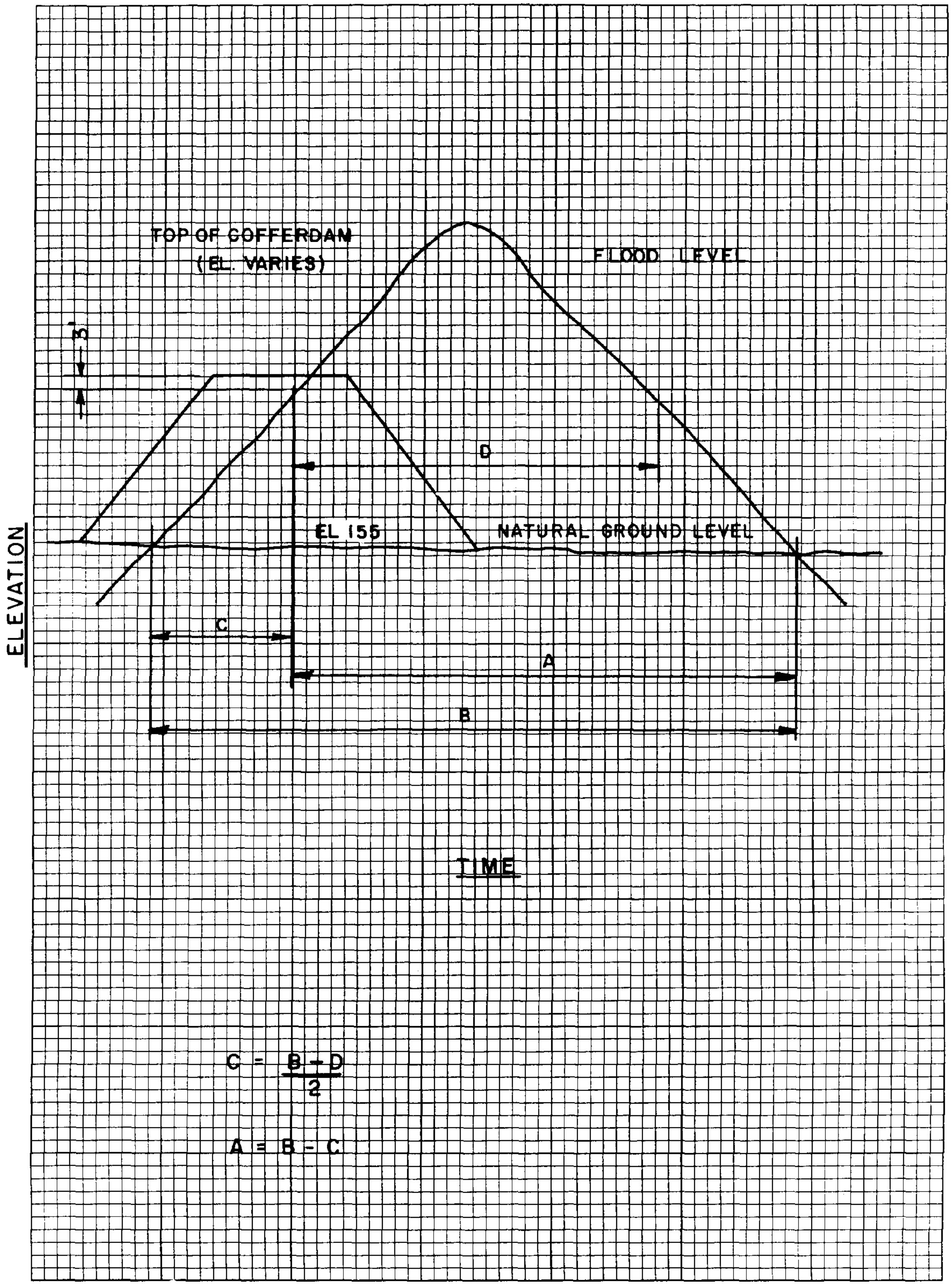


Figure 6-8. Method of duration analysis

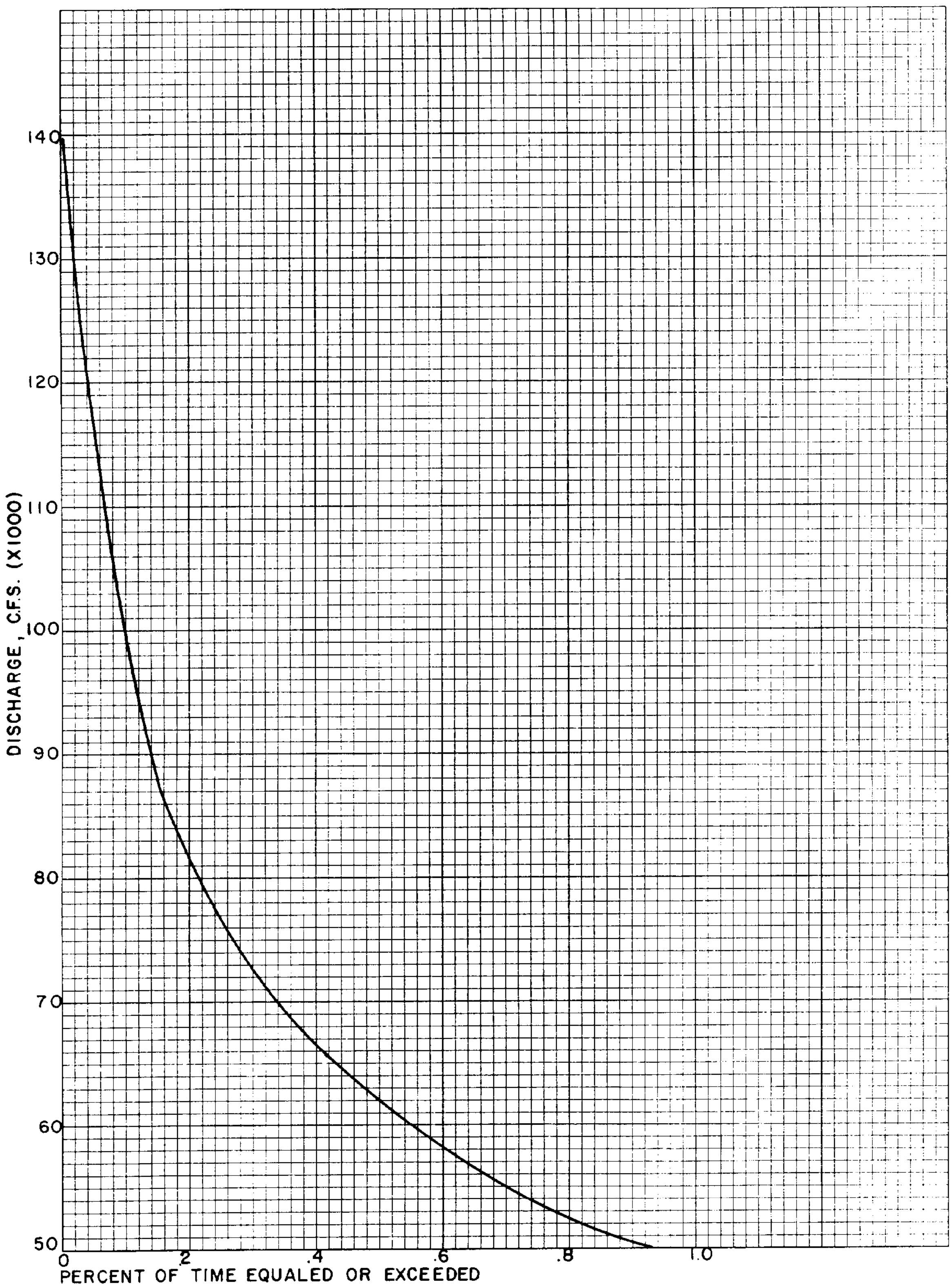


Figure 6-9. High discharge duration curve

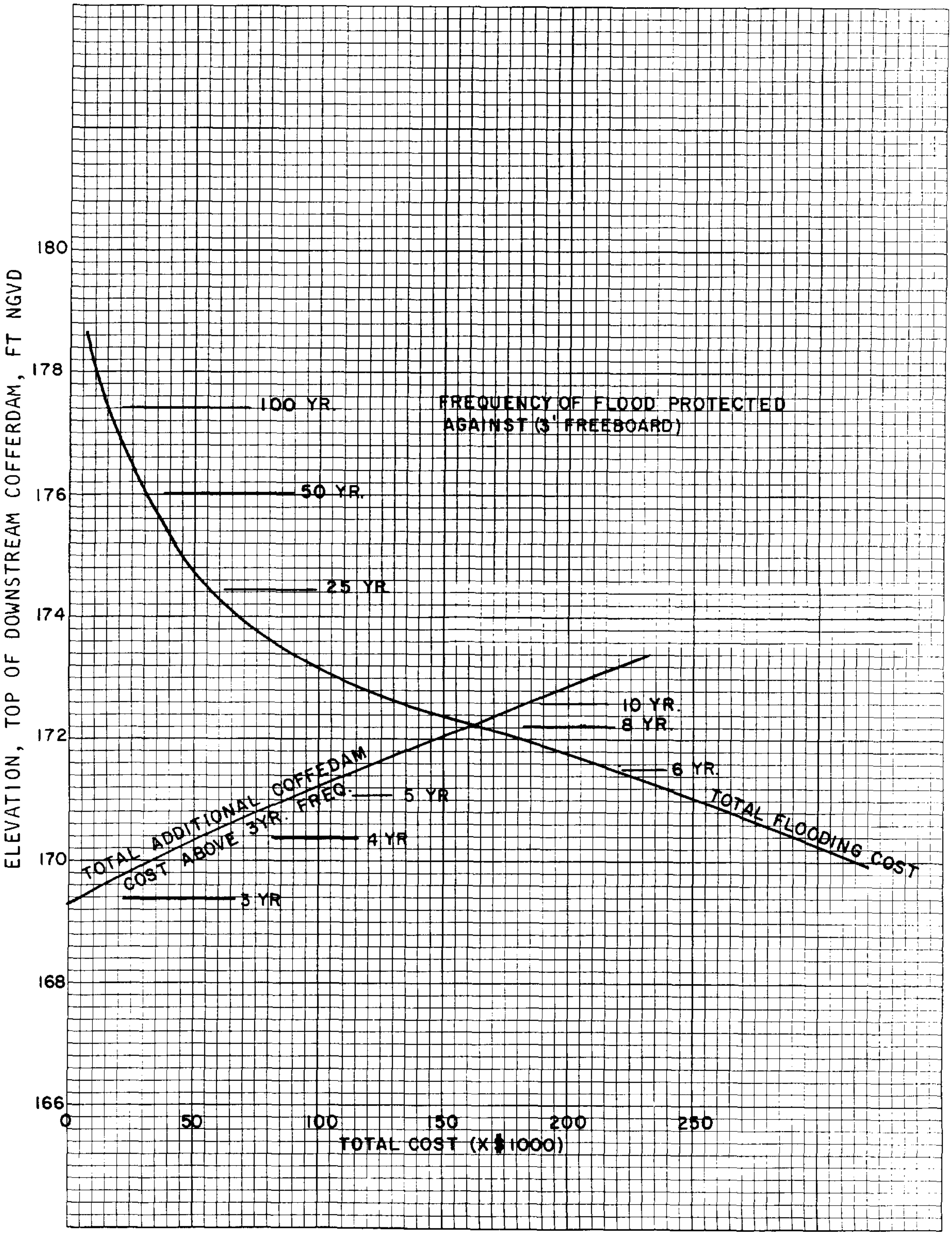


Figure 6-10. Cofferdam and flooding cost curves

flooding cost is compared with the total additional cofferdam cost required to provide protection above the three-year frequency flood level. Visual inspection of the curves indicates that the most economical cofferdam elevation will be near the 10-year flood level. It should be noted that the intersection of the two curves in Figure 6-10 has no significance because the beginning ordinate of the cofferdam cost curve is arbitrary. In Figure 6-11, the probable flooding cost reduction and the additional cofferdam costs were established by determining the slope of the total cost curves at incremental cofferdam heights. The curves show the rate of change in probable flooding cost reduction and the additional cofferdam cost for various cofferdam top elevations. The upper intersection between the two cost curves in Figure 6-11 represents the point of diminishing returns. In this example, the point is at elevation 172.9 which was arbitrarily rounded to 173.0. The design flood frequency was therefore set at 12 years.

6-8. 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 to assist in 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.

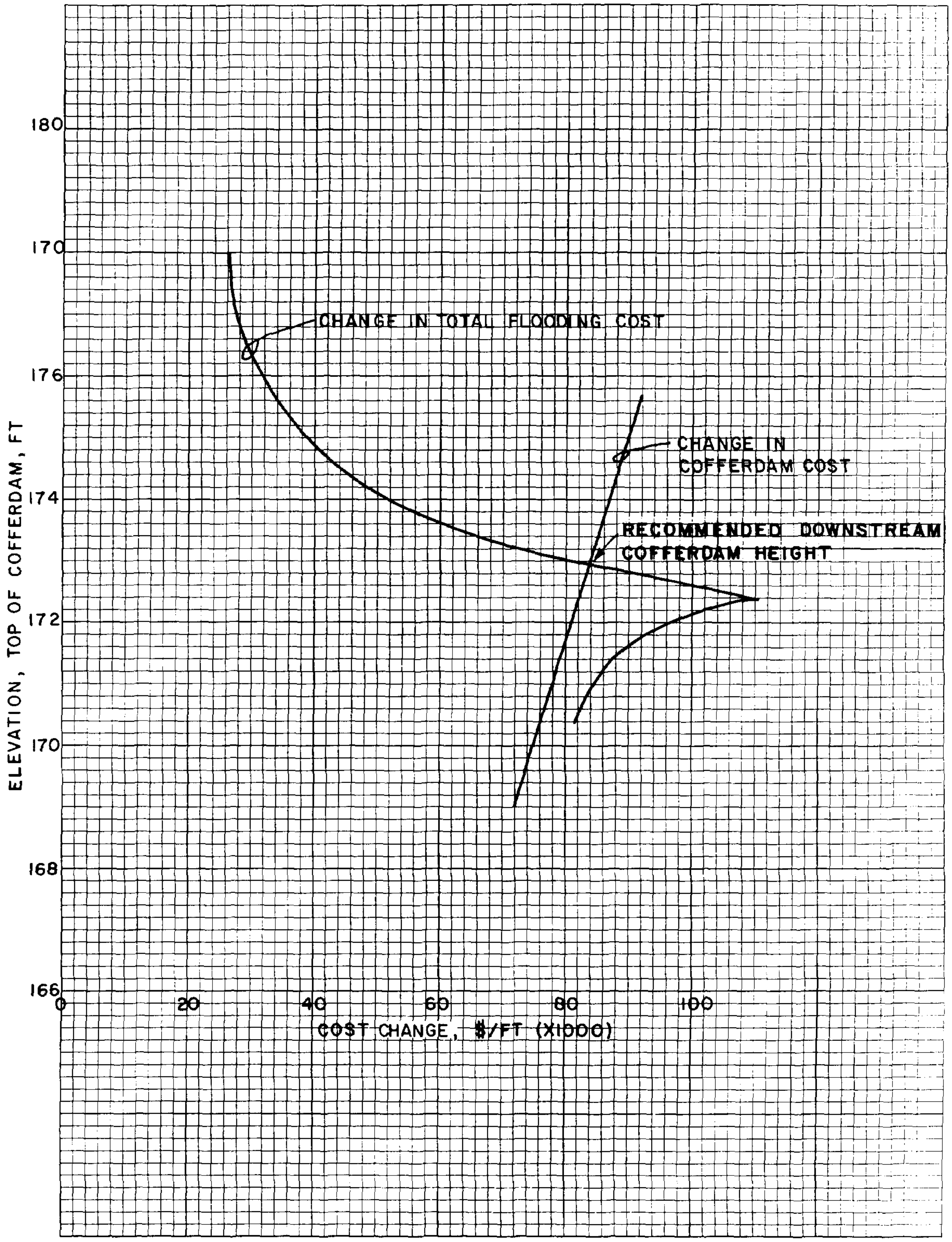


Figure 6-11. Cofferdam and flooding cost change curves

CHAPTER 7

PROJECT OPERATION

Section I. Normal Spillway Operations

7-1. Maintenance of Navigation Pool Levels. The purpose of maintaining a navigation pool on a river navigation project is to assure 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 above purpose is met is usually defined as the "normal pool."

a. Uncontrolled Spillways. These structures consist basically of a fixed-crest weir; a typical example is shown in Figure 7-1. 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.

b. Gated Spillways. The normal pool elevation, consistent with its definition in paragraph 7-1, is maintained by the operation of dam gates. It should be noted that in 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 (see paragraph 7-3b), 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 will be prepared for flooding. A gate operation schedule should be prepared during the design stage. An example of Pittsburgh District's gate operation schedule for the Maxwell Lock and Dam on the Monongahela River is shown in Figure 7-2. 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

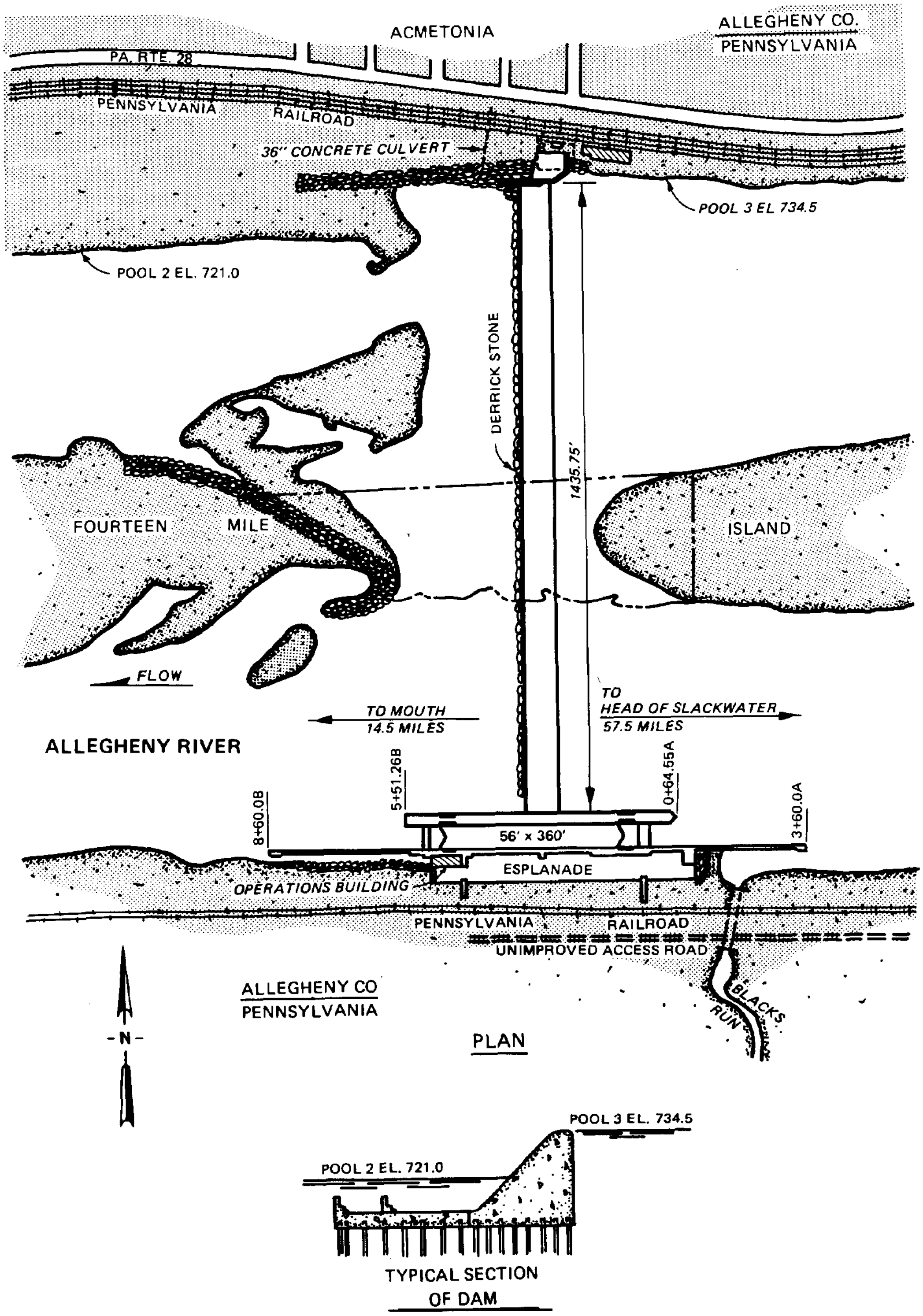


Figure 7-1. Uncontrolled spillway

NORMAL UPPER POOL EL. 763 (GAGE 9.0)
MINIMUM LOWER POOL EL. 743.5 (GAGE 9.0)

Discharge cfs	Lower Pool Elev ft msl	Lower Pool Gage ft	Double Leaf 1	Single Leaf 2	Single Leaf 3	Double Leaf 4	Double Leaf 5	Total Feet Open	Upper Gage in Feet Dam No. 7
0	743.5	9.0	GATES CLOSED	GATES CLOSED	GATES CLOSED	GATES CLOSED	GATES CLOSED		9.0
2,900	743.8	9.3			1			1	10.3
5,300	744.0	9.5			2			2	11.0
8,150	744.5	10.0			2	1		3	11.6
10,800	744.9	10.4		1	2	1		4	12.0
16,100	745.8	11.3	1	1	2	1	1	6	12.9
22,200	747.1	12.6	1	2	2	2	2	9	13.6
27,400	748.3	13.8	2	2	4	2	2	12	14.2
33,700	749.6	15.1	2	4	4	4	2	16	14.9
39,300	750.8	16.3	4	4	4	4	4	20	15.4
45,500	752.1	17.6	4	4	6	6	4	24	16.0
50,900	753.3	18.8	4	6	6	6	6	28	16.4
55,400	754.4	19.8	6	6	8	6	6	32	16.8
59,400	755.1	20.6	6	8	8	8	6	36	17.1
63,000	755.8	21.3	8	8	8	8	8	40	17.4
66,800	756.4	21.9	8	8	10	10	8	44	17.8
69,500	757.1	22.6	8	10	10	10	10	48	18.1
72,000	757.6	23.1	10	10	12	10	10	52	18.3
74,100	758.0	23.5	10	12	12	12	10	56	18.6
76,000	758.4	23.9	12	12	12	12	12	60	18.8
78,100	759.0	24.5	12	12	14	14	12	64	19.0
80,100	759.4	24.9†	12	14	14	14	14	68	19.2
83,800	760.2	25.7	14	14	GATES RAISED CLEAR OF WATER	14	14	78*	19.6
87,400	761.0	26.5	14	14	GATES RAISED CLEAR OF WATER	GATES RAISED CLEAR OF WATER	14	86*	20.1
89,700	761.5	27.0	14	GATES RAISED CLEAR OF WATER	GATES RAISED CLEAR OF WATER	GATES RAISED CLEAR OF WATER	14	94*	20.4
92,400	762.0	27.5	GATES RAISED CLEAR OF WATER	GATES RAISED CLEAR OF WATER	GATES RAISED CLEAR OF WATER	GATES RAISED CLEAR OF WATER	GATES RAISED CLEAR OF WATER	110*	20.7

No. 1 gate is next to lock. Gate openings are shown below the gate numbers; these openings are in feet from crest of dam to the bottom of gates. Any operation step may be made in parts for closer control. Two feet is the maximum desirable difference in opening of adjacent gates.

* Effective opening raised clear of water assumed to be 22 feet. Gage readings at Dam 7 (upper) correspond to discharges shown at Maxwell.

† Desirable minimum tailwater for 1 gate fully open.

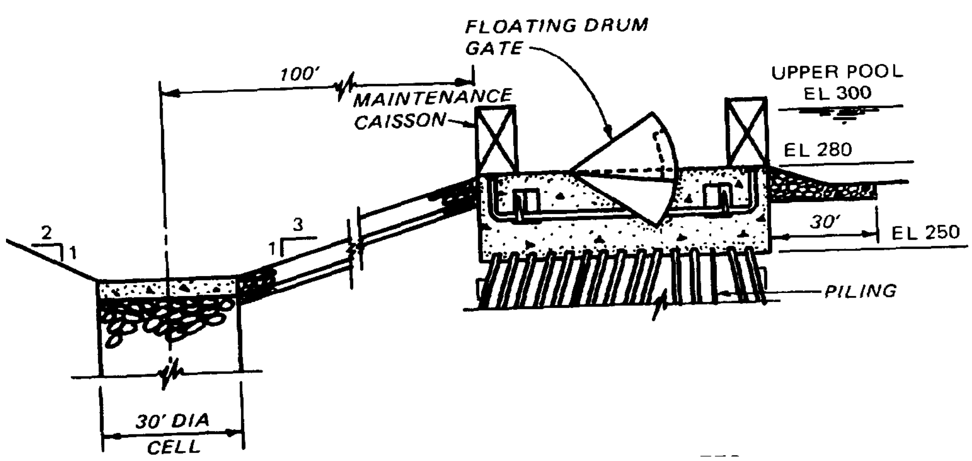
Figure 7-2. Gate operation schedule for Maxwell Lock and Dam uniform distribution of gate openings across the structure to prevent the formation of dangerous eddies downstream. Finally, the attainment of low operation costs and enhancement of water quality at low flows are also important operating objectives. 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).

c. 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 assure sufficient depth for navigation. Movable dams are structures that accomplish this objective. An early version of movable dams were 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. A typical wicket dam is shown in Figure 5-18. The operation of the wicket dams in their original form is rather time-consuming and hazardous, especially during winter periods. Therefore this type of operation can be considered obsolete. An improved version of the wicket dam concept, utilizing remotely controlled hydraulic cylinders, has been built recently on the Seine River in France. A more modern type of movable dam has been proposed for the navigable pass portion of the single dam replacement structure at Olmsted on the lower Ohio River (Figure 7-3). For the preliminary design, a drum gate that is raised or lowered by the upstream hydraulic pressure was considered for the movable portion of the dam. The control is remote to eliminate any hazardous manual operation.

7-2. Low-Flow Periods. The operation of movable dams to ensure navigation depth during low-flow periods has been described in the previous paragraph. 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. An example of this operation is shown in Figure 7-4. 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, etc. 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.

7-3. Flood Flow Periods.

a. High Dams. Navigation projects with high dams are usually



SECTION THRU DRUM GATES

Figure 7-3. Movable dam

Gates 1	2	3	4	5	6	7	8	Approx Discharge cfs
				1				3,000
				2				6,100
				3				9,300
				4				12,500
				5				15,600
		1		5				18,400
		2		5				21,400
		3		5				24,400
		5		5				30,600
		5		5		1		33,200
		5		5		3		39,000
		5		5		5		44,700
		5	1	5		5		47,200
		5	3	5		5		52,600
		5	5	5		5		58,000
		5	5	5	1	5		60,300
		5	5	5	3	5		65,200
		5	5	5	5	5		70,200
	1	5	5	5	5	5		72,400
	3	5	5	5	5	5		77,100
	5	5	5	5	5	5		81,600
	5	5	5	5	5	5	1	83,700
	5	5	5	5	5	5	3	88,000
	5	5	5	5	5	5	5	92,400
1	5	5	5	5	5	5	5	94,200
3	5	5	5	5	5	5	5	98,300
5	5	5	5	5	5	5	5	100,400

Note: Gate openings in feet.

Figure 7-4. Gate operating schedule for improved reaeration,
Racine Locks and Dam

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 feet. 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 which is not used for hydropower generation. An example of a multipurpose high navigation dam is the Wheeler project on the Tennessee River operated by TVA (Figure 7-5). During flood periods, spillway gates are operated to pass flood flows 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. As shown in Figure 7-6, the project is also operated for flood control by drawing the pool down to el 549 in anticipation of spring floods. The minimum pool is established by providing for authorized navigation depth.

b. 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.

c. 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

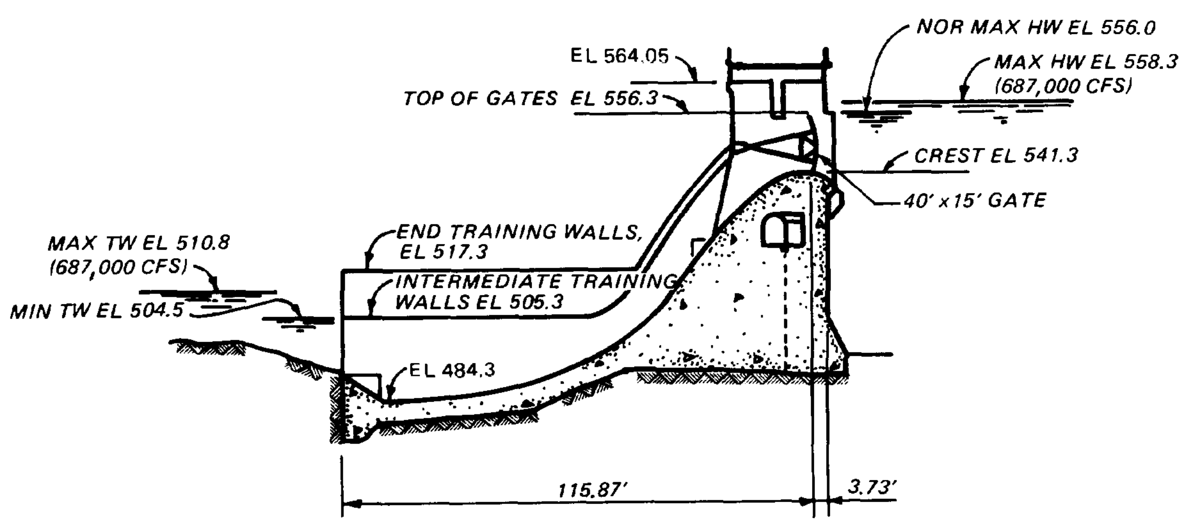
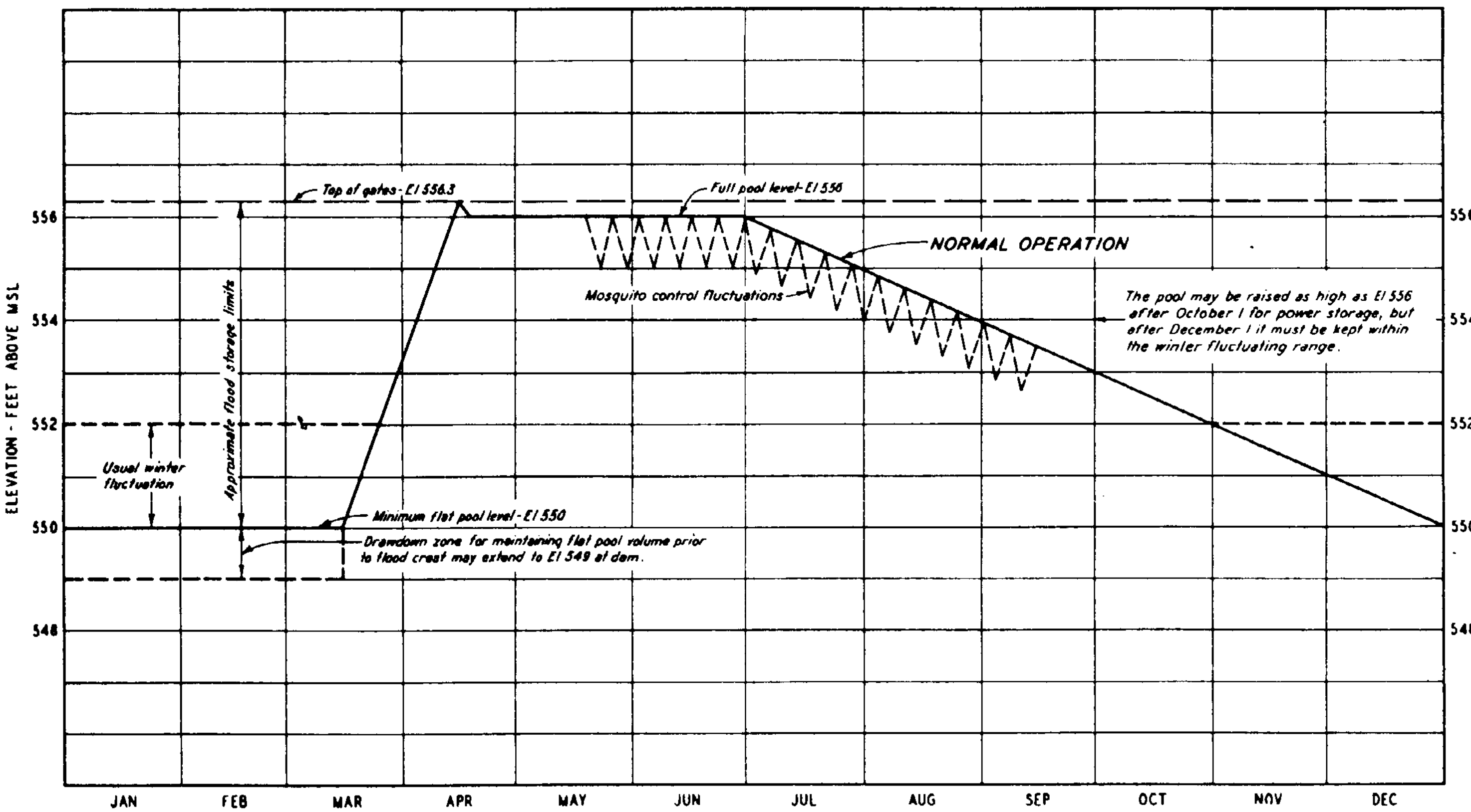


Figure 7-5. Multipurpose navigation project, Wheeler Dam, Tennessee



NOTES:
Elevations apply only at dam.
Maximum level assumed for design of dam - El 558.3

Figure 7-6. Operating curve for pool elevations, Wheeler Dam, Tennessee 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.

(1) Stage Control.

(a) 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. Additionally , control stage limits exist at certain point or points within the pool that must not be exceeded for these range 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.

(b) Example. In the pool of Dam No. 26 on the Mississippi River, a nine-foot-deep navigation channel must be maintained during flow periods of 210,000 cfs or less. Additionally, stages at Grafton, Ill., approximately 15 miles upstream of Dam No. 26 must not exceed 420.0 feet NGVD. During minimum flows, the pool level at the dam is maintained at 419.0 feet 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, Ill. When approaching a discharge of 210,000 cfs, the pool at the dam must be lowered to 414.0 feet NGVD to accomplish the above 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 five feet exists at the dam (419.0 to 414.0 feet NGVD) as discharges increase from minimum flows to those providing uncontrolled navigation depth.

(2) Real Estate Acquisition.

(a) Purpose. For some projects, hinging the pool can reduce the required amount of flowage easement acquisition because of lowered post-project flow-line profiles throughout the pool.

(b) 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 7-7 illustrates the flow-line reductions that can be realized by hinging this pool.

(3) Pool Dredging Quantities.

(a) Purpose. During the recession period of flood flows, sediments tend to deposit 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

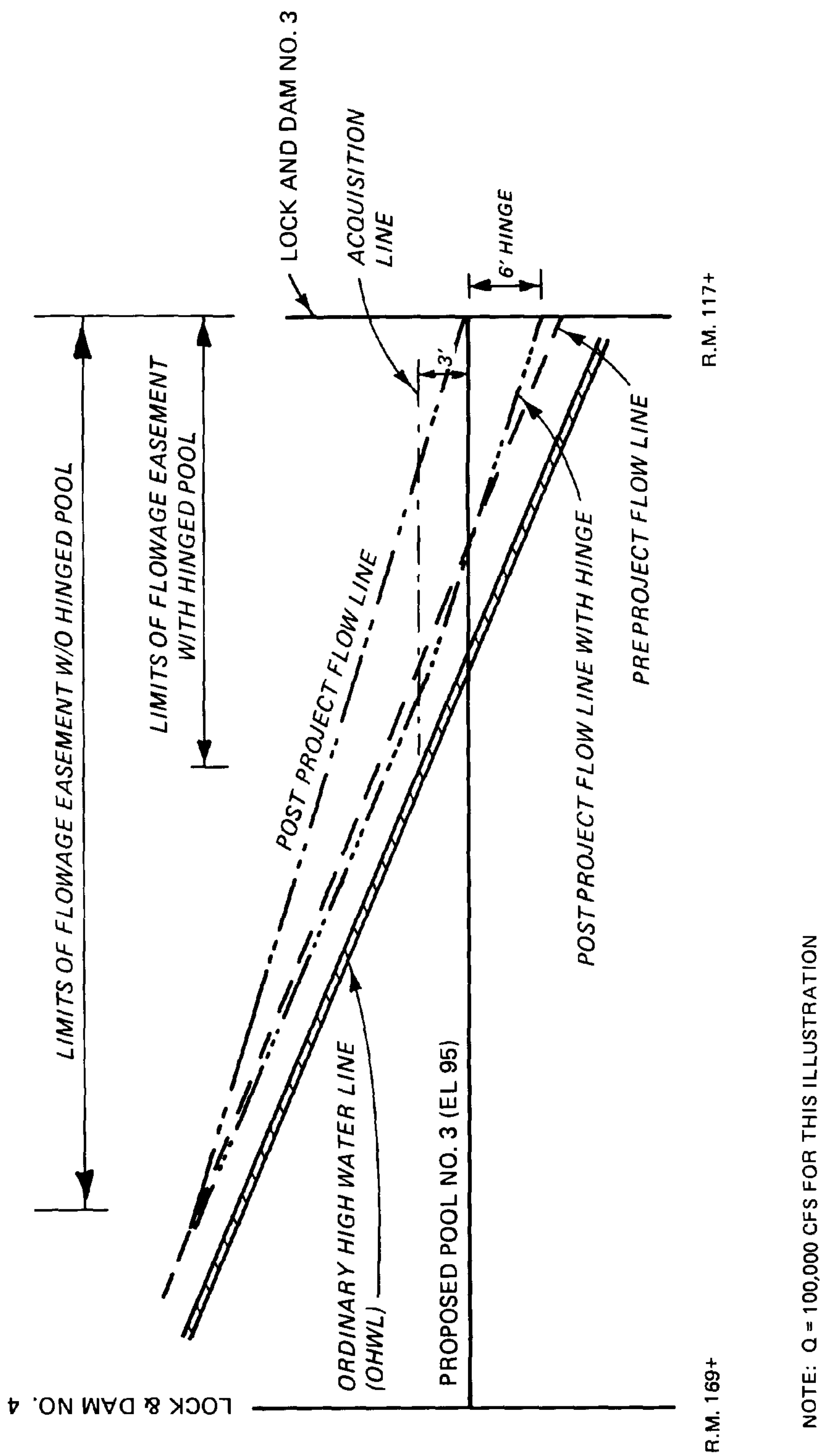


Figure 7-7. Hinged pool operation

farther downstream in the pool where postflooding depths are adequate for navigation without dredging.

(b) 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.

(4) Hinged Pool Consequences. If hinged pool operations are anticipated on a navigation pool, several factors must be considered in the project design.

(a) 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.

(b) Velocities and crosscurrents in the upper lock approach will be more severe than under normal navigation pool conditions.

(c) Tie-up facilities along the lock guide and guard walls must be usable at the lowered pool levels.

(d) 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.

(e) Sudden pool drawdowns can result in bank instabilities.

(f) The increased complexity of operating the spillway gates for the hinged pool levels can lead to misoperations.

7-4. Ice and Debris Passage.

a. 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 EM 1110-2-1612 for additional information.

b. 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 condition and experience; it is usually one-third to fully opened gate depending on tailwater level (see items 15 and 18). 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 above. 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 in the North Central Division on the Illinois Waterways and the Mississippi River, respectively. Submergible tainter gates are proposed for several projects on the Illinois River and model studies will be conducted to ensure vibration-free operation.

c. 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.

d. 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.

Section II. Special Spillway Operations

7-5. 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.

7-6. 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. The responsible individual will be required to decide on spillway operations that are in the best overall interest of the project, considering project protection, navigation needs, and safety.

7-7. 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, as discussed in Section 7-6. Misoperation can cause abrupt changes in upper pool and tail-water elevations. It may also cause problems at adjacent locks or hydro-electric 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 responsible individual will need to have the gate settings corrected as soon as possible after the misoperation is discovered. The recovery operation must be executed so that abrupt changes in stage that could cause problems are not created. A survey for damage should be conducted as soon as practicable after the recovery.

7-8. Equipment Malfunction. Many types of equipment malfunctions may require special operations in order to recover normal capability. Some examples are covered below. In any case, the responsible individuals will need to analyze the particularities of each case, and plan and execute necessary operations and repair, in order to return the facility to normal operational status while minimizing the impact on project functions during the recovery period.

a. 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.

b. 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. The responsible individual will need to know if the gate load is equally distributed on each side of the gate. If not, 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. Additional steps, as in paragraph a. above, may be required.

c. Equipment Vibrations. Flow-induced vibrations have the potential for causing considerable damage to gates and other equipment. Vibrations are discussed in Chapter 5. Vibrations can vary from the nuisance level to a major, structurally damaging problem. Regardless of the perceived seriousness of the problem, vibrations observed by operating personnel should be brought to the attention of higher authority for evaluation. 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. This may require additional gate changes, as in paragraph a. above, or other operational modifications appropriate to the instant circumstances.

7-9. 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.

7-10. Emergency Operation.

a. 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.

b. 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 7-8 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, including loss of the navigational pool. Serious accidents are more likely to occur during high-water periods than during low water. Designers and operators should be aware of those conditions that are more likely to cause serious damage to the structure in case of collision. 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. In the process of developing an operating plan, the responsible individuals may want to require a staggered gate operation in order to reduce the potential for a current concentration approaching the spillway (e.g., gates 2, 4, and 6 should be raised one increment followed by raising gates 1, 3, and 5).

c. Emergency Closure. Two types of closure devices are common:

(1) Bulkheads. The most common type of emergency closure for spillway gate bays is a bulkhead consisting of one or more sections and

Figure 7-8. Accident at Maxwell Lock and Dam, Ohio River

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. The vertical height of the structure may vary from three to twelve feet depending on design constraints of a specific project. 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 on 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.

(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. It should be noted

that the bulkheads described in the previous paragraph are sometimes designated as stop logs.

d. Drawdown. Requirements for low-level discharge facilities for drawdown of impoundments are given in EM 1110-2-50. Such facilities may also provide flexibility in future project operation for unanticipated needs, such as major repairs of the structure, environmental controls, or changes in reservoir regulation.

CHAPTER 8

REPAIR AND REHABILITATION

8-1. 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. Specific repair and rehabilitation methods are presented in the REMR notebook (item 27).

8-2. 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. Al though the design life of most projects is 50 years, the practical usable life is much longer.

8-3. 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 the following:

a. Modern machinery.

b. Modern electrical equipment.

c. Remote controls.

d. Television surveillance system including audio in some instances.

e. Emergency closure.

f. Adding gates to ungated spillways.

8-4. Typical Repair and Rehabilitation Items. The following are common items for major navigation dam rehabilitation projects:

a. Dam Stability.

(1) Replace upstream and downstream scour protection.

(2) Tendons through structure into foundation.

(3) Cutoff of dam underseepage.

b. Discharge Capacity.

(1) Additional gates.

(2) Overflow dikes.

(3) Raise dam.

c. Ice and Debris Control.

(1) Submerged gates.

(2) Control booms.

(3) Air screens.

(4) Gate heaters.

d. Replacement in Kind.

(1) Resurface concrete surfaces.

(2) Repair or replace gates.

(3) Fix gate anchorages.

(4) Replace embedded metal.

(5) Electrical and mechanical equipment.

8-5. Scour Protection.

a. 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.

b. Existing Project Design. Repair of existing projects requires evaluation of the same conditions listed in paragraph 8-4. 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.

c. Consequence of Failure. An analysis of the consequences 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.

d. 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 impacts. Design innovations based upon sound judgment that are well documented are encouraged.

e. 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.

f. 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 structure is 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.

g. 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.

8-6. Repair and Rehabilitation Model Studies. The following model studies for major rehabilitation have been conducted by WES to address repairs to scour protection:

Project	Feature	Problem	Recommendation
Arkansas River Dams	Spillway gates	Gate vibrations	Remove seals on the bottom of gates. Projects requiring bottom gate seals should use Type D in Figure 5-19

Project	Feature	Problem	Recommendation
Cheatham Dam	Spillway gates	Modify partially submergible gates to lift gates	Retain original gates and modify the sill and trajectory (Add 1.2 feet to sill elevation and an $x^2 = 26.8y$ trajectcry 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 forty years of operation	Provide additional scour protection by underwater placement of quarrystone and graded riprap as determined in model tests
Montgomery Dam, Ohio River	Scow repair down-stream 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 begs
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 saw	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

APPENDIX A

BIBLIOGRAPHY

1. American Society of Mechanical Engineers. 1958. "American Standard Letter Symbols for Hydraulics," ASA Y10-1958, New York.

2. Basco, D. R. 1970 (May). "An Experimental Study of Drag Force and Other Performance Criteria of Baffle Blocks in Hydraulic Jumps," MP H-70-4, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

3. Cox, R. G. 1972 (Jun). "Pertinent Data on Spillway Tainter Gates for Corps of Engineers Projects," MP H-72-7, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

4. Glenn, H. T. 1976 (Apr). "Interim Report on Tainter Vibrations at Barkley Dam" (unpublished report), US Army Engineer District, Nashville, Nashville, Tenn.

5. Grace, J. L. 1964 (Jun). "Stability of Riprap and Discharge Characteristics, Overflow Embankments, Arkansas River, Arkansas," TR 2-650, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

6. Grace, J. L. 1964 (Sep). "Spillway for Typical Low-Head Navigation Dam, Arkansas River, Arkansas," TR 2-655, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

7. Hart, E. D., and Hite, J. E., Jr. 1979 (May). "Prototype Gate Vibration Tests," TR HL-79-8, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

8. Hite, J. E., Jr., and Pickering, G. A. 1983 (Aug). "Barkley Dam Spillway Tainter Gate and Emergency Bulkheads, Cumberland River, Kentucky," TR HL-83-12, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

9. Jones, H. W. 1977 (Jun). "Manual for a Conversationally Oriented Real-Time Program-Generating System (CORPS)," Unnumbered Report, US Army-Engineer Waterways Experiment Station, Vicksburg, Miss.

10. Markussen, J. V. "Red River Lock and Dam No. 2, Draft Report, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

11. Maynord, Steve T. (1986). "Open Channel Riprap Design," Proceedings of the Third International Symposium on River Sedimentation, Jackson, Miss.

12. Ortaono, L. 1974 (Jan). "Issues in Water Resources Impact Assessments," Journal, Hydraulics Division, American Society of Civil Engineering, Vol 100, No. HY1, Proc Paper 10292, pp 173-187.

13. Oswalt, N. A. 1974 (Oct). "Spillway for Aliceville Lock and Dam, Tombigbee River, Alabama, TR H-74-10, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

14. Oswalt, N. R. 1977 (Jun). "Red River Waterway, La., Tex., Ark., and Okla., Mississippi River to Shreveport, La., Reach, Lock and Dam No. 1," TR H-77-13, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

15. Oswalt, N. R., and Pickering, G. A. 1973 (Oct). "Spillway for Lock and Dam 26, Mississippi River, Missouri and Illinois," TR H-73-15, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

16. Oswalt, N. R., and Pickering, G. A. 1974 (Nov). "Spillway for Columbus Lock and Dam, Tombigbee River, Alabama, " TR H-74-13, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

17. Pickering, G. A. 1971 (Jun). "Spillway Gate Vibrations on Arkansas River Dams, Arkansas and Oklahoma, " TR H-71-5, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

18. Pickering, G. A., and Grace, J. L., Jr. 1965 (Dec). "Spillway for: Cannelton Locks and Dam, Ohio River, Kentucky and Indiana," TR 2-710, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

19. Price, James T. 1965 (Apr). "Flow-Induced Vibrations - Experience of TVA with Hydraulic and Other Structures," Civil Engineering, Vol 35, No. 4, p 61.

20. Pugh, C. A. 1977 (Apr). "Spillway Vibration, Pressure and Velocity Measurements, Ozark Lock and Dam, Arkansas River, Arkansas," TR H-77-6, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

21. Schmidgall, Tasso. 1972 (Jan). "Spillway Gate Vibrations on Arkansas River Dams," Journal of the Hydraulics Division, ASCE, Vol 98, No. HY1.

22. Schmitt, Robert W. 1969 (Aug). "Simulation of Flow Through Broad Crest Navigation Dams, " Thirteenth Congress of the International Association of Hydraulic Research, Vol 1, Subject A, Japan.

23. Simmons, William P. 1967 (Mar). "Experiences with Flow-Induced Vibrations: Journal of the Hydraulics Division, Vol 91, No. HY 4, p 185, 6 Jul, 1965 and CLOSURE thereto in Vol 93, No. HY 2, p 45.

24. Thomas, William A. 1969 (Mar). "Interim Report on Vibration of Spillway Gates" and Supplement No. 1, "Natural Frequencies" (unpublished report), US Army Engineer District, Little Rock, Little Rock, Ark.

25. Thomas, William A. and McAnally, William H., Jr. "Users Manual for the Generalizer Computer Program System Open-Channel Flow and Sedimentation Tabs-2" Department of the Army, Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss.

26. US Army Corps of Engineers. 1981 (Dec). "The Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251," Final Report to Congress.

27. US Army Corps of Engineers, Waterways Experiment Station. 1985. "The REMR Notebook," Vicksburg, Miss.

28. US Army Corps of Engineers, Water Resources Support Center, "HEC-6 Scour and Deposition in Rivers and Reservoirs, Users Manual," The Hydrological Engineering Center, Davis, Calif.

29. US Army Engineer District, Little Rock. 1960 (May). Arkansas River and Tributaries, Arkansas and Oklahoma, Project Design Memorandum No. 5-3, "Navigation Channel Appurtenances, Normal Pool Elevation and Dam Sites"; Little Rock, Ark.

30. US Army Engineer District, Louisville. 1985 (Jun). "Submergible Gate Use Within the Corps," Louisville, Kentucky.

31. US Army Engineer Institute for Water Resources, CE. 1972 (Jun). "An Analysis of Environmental Statements for Corps of Engineers Water Projects," IWR Report 72-3, Fort Belvoir, Va.

32. US Army Engineer Waterways Experiment Station. 1961 (Dec). "Swellhead Effect of Navigation Structures," MP No. 2-468, Vicksburg, Miss.

33. Working Committee on Federal Construction, Washington, DC. 1968 (1 Nov). "Report to the Vice-President and to the President's Council on Recreation and Natural Beauty."

Selected References

Petersen, Margaret S. 1986. "River Engineering," Prentice-Hall.

Schumm, Stanley A. "The Fluvial System," John Wiley & Sons, New York.

Shen, H. W. 1971. "River Mechanics," Colorado State University, Fort Collins, Colorado.

Simons, Li, & Associates. "Engineering Analysis of Fluvial Systems," Colorado State University, Fort Collins, Colorado.

Straub, L. G. 1935. "Missouri River Report," House Document 238, Appendix SV, US Army Corps of Engineers, to 73rd US Congress, 2nd Session.

Toffaleti, F. B. 1969 (Jan). "Definite Computations of Sand Discharge in Rivers," J of the Hydraulic Div., ASCE Vol 95, No. HY 1, pp 225-246.

Vanoni, V. A. Ed. 1975. "Sedimentation Engineering, ASCE Manual 54," American Society of Civil Engineers, New York.

APPENDIX B

NOTATION

Symbol	Term	Units
A	Cross-sectional area	ft^2
B	Crest elevation - stilling basin apron elevation	ft
B_c	Width of horizontal portion of broad-crested weir	ft
C	Discharge coefficient (subscript denotes type) Isbash coefficient Cost or losses (subscript denotes type)	-- -- $
C_G	Top of gate elevation - crest elevation	ft
d	Depth	ft
d_1	Depth before hydraulic jump	ft
d_2	Depth after hydraulic jump	ft
D	Number of days construction area is flooded before cleanup operations can begin	--
D_{50}(MAX) D_{50}(MIN)	Riprap diameter at which 50 percent is finer by weight. MAX and MIN refers to the upper and lower limits for the allowable gradation.	ft
D_{100}	Maximum riprap size	ft
D_G	Drop control = top of gate elevation - lower pool elevation.	ft
F	Froude number = $V/\sqrt{gd}$ (subscript denotes type)	--
g	Gravitational acceleration	ft/sec^2
G_O	Gate opening = vertical distance between gate lip and spillway crest.	ft
H	Energy head on spillway crest = upper pool elevation + $V^2/2g$ - crest elevation	ft
h	Height of tailwater above spillway crest	ft
H_g	Head on gate q H - $G_o/2$	ft
h_b	Height of baffle blocks	ft

Symbol	Term	Units
i	Number of event	--
k	Spillway coefficient of contraction in d'Aubuisson equation	--
L	Length of spillway crest	ft
L_1	Distance from beginning of stilling basin to upstream face of the first row of baffles	ft
L_2	Distance from beginning of stilling basin to beginning of end-sill upslope	ft
N	Number of trials	--
P	Probability	--
P	Probability, approach depth	--
Q	Discharge	cfs
q	Unit discharge	cfs/ft
R	Radius	ft
SUb_G	Gate submergence = upper pool elevation - top of gate elevation	ft
TW	Tailwater elevation	ft, NGVD
V	Average velocity = Q/A (subscript denotes type)	ft/sec
V_o	Initial free jet velocity	ft/sec
W_{50}	Riprap weight at which 50 percent is finer by weight	lb
X	Horizontal or longitudinal coordinate or distance	ft
Y	Vertical or transverse coordinate or distance	ft
AH	Difference between headwater and tailwater elevations = H - h	ft
γ_w	Unit weight of water	lb/ft^3
γ_s	Unit weight of stone (saturated surface dry)	lb/ft^3
π	Constant = 3.1416	--

APPENDIX C

NAVIGATION DAM MODEL AND PROTOTYPE STUDY DATA

1. Introduction. The availability of data from Corps of Engineers hydraulic model and prototype investigations of navigation dams is summarized in Table Cl. This information was obtained from a detailed review of 120 reports on model and prototype studies (1930 to 1984) by the St. Paul District, Bonneville Hydraulic Laboratory, and Waterways Experiment Station. These reports are listed in the accompanying bibliography. The organization and use of Table Cl are described in the following paragraphs. The data were not analyzed or evaluated with regard to quality, present design practice, etc.

2. Design and Operational Variables. A list of 221 hydraulic design and operational variables or significant features of navigation dams was derived from a review of such items in various designs of dams used at CE locks. This list is organized in an upstream-to-downstream order and has a numbering sequence for easier manipulation in a digital computer. The major divisions of the list include:

21000 UPSTREAM APPROACH
22000 CONTROL SILL
23000 GATES AND BULKHEADS
24000 STILLING BASIN (APRON)
25000 DOWNSTREAM CHANNEL

A listing of operational variables is included with each major division in Table Cl rather than in a separate division in order to group more closely the aspects of the dam operation with their related design features. The 20 "NOTED ITEMS" include special items peculiar to the specific projects and are identified in the notes at the end of Table Cl.

3. Test Reports. Each column heading in Table Cl includes a very brief identification of the project and a brief notation of the report number (full title in the Bibliography to this Appendix). All of the 120 reports are available on loan from the WES Technical Library. The initial letter rather than number characters in the column numbers (A01 to B21) was used for easier identification in a digital configuration for computer file manipulation.

4. Types of Data in Reports. The types of performance data available in each report and pertaining specifically or generally to the various design and operational features investigated are indicated by the following letter symbols in Table Cl:

T - time-related data

Q - discharge, including coefficients

U - stilling basin performance, flow regime, appearance

H - hawser force on tow

D - tow displacement, unrestrained by hawsers

V - local velocities (surface, internal, bottom)

C - surface currents, including vortices

N - effects on navigation

B - boils, or surface turbulence

W - waves

Y - water-surface elevation profile

S - surges or oscillations

I - internal flow pattern or flow distribution

E - erosion pattern, profile or depth

R - riprap performance (scour, stability)

Z - local average piezometric pressures

P - local transient or fluctuating pressures

L - losses or differences (head, pressures)

F - mechanical forces or torque

A - vibration, bouncing

X - other data (see last line of NOTED ITEMS at end of Table Cl)

5. Comments. The following comments result from observations during the compilation of Table Cl and may be of interest and/or assistance to users searching for available test data pertinent to their design problems.

a. Consideration of both the design and operational variables of the feature under investigation, both more general and more specific identification of the variables, and related items or systems in Table Cl may aid in finding applicable data that might otherwise be missed.

b. The listing of operational variables at “division level” in Table Cl and the compilation process may have resulted in some inappropriate entries of types of data relative to design variables. This would most likely occur where a report table or illustration includes several kinds of design and operational variables.

c. Variables 24200 Apron, 25100 Channel, and 25121 Invert El were given data references for most of the citations involving spillway performance. Although there may not have been any design variations in the apron or channel, they are locations of primary interest for most aspects of spillway operation.

d. Studies of a few nonnavigation dams were included in the listings because of those projects' similarity to navigation dams in general design and/or operation. Some data on fishways and construction cofferdams were noted if such were included in the reports, but all available studies on these items were not reviewed.

6. Detailed Test Data Listings. The LINE NO'S correspond to those 221 numbers assigned to the design and operation variables. The TYPE OF DATA symbols correspond to those given in paragraph 4 above. The FORMAT symbols are:

T - numbered tables

P - numbered photographs

D - numbered drawings (plates)

F - numbered figures (covers all illustrations in St. Paul District reports)

W - text paragraphs (or pages if unnumbered paragraphs) containing information not indicated by the tables, photographs, drawings, or figures.

The LOCATION IN REPORT numbers and letters are those of the pertinent tables, photographs, drawings, figures, and/or paragraphs in that particular report.

7. In addition to the indicated tables, photographs, drawings, and/or figures having data pertinent to a specific design and/or operational variable, the user should refer to those parts of the text where these data items are discussed. The comment in subparagraph 5b above also applies to the detailed data listings. Also, variations in design and/or operational variables from table to table, photograph to photograph, etc., rather than in individual tables, photographs, etc., are covered by listings of all the related data item location numbers. The user should compare variables from item to item as well as in a single item.

8. A total of 20,067 location citations was derived from a total of 4,930 single- or combined-item references (tables, photographs, drawings, figures, text) in the 120 reports. The item location numbers are referenced in the Bibliography to Appendix C (A01, B01, etc.)

TABLE 1
NAVIGATION DAM
MODEL AND PROTOTYPE STUDY DATA

PAGE SEQUENCE FOR TABLE 1

DESIGN AND OPERATIONAL VARIABLES	TEST REPORT COLUMN NUMBERS					
	A01 TO A20	A21 TO A45	A46 TO A65	A66 TO A90	A91 TO B11	B12 TO B36
21000 TO 21990	1	2	3	4	5	6
22000 TO 22990	7	8	9	10	11	12
23000 TO 23990	13	14	15	16	17	18
24000 TO 24760	19	20	21	22	23	24
24900 TO 25400	25	26	27	28	29	30
25900 TO 25990 AND "NOTED ITEMS"	31	32	33	34	35	36
	FACING PAGES		FACING PAGES		FACING PAGES	

1. SELECT DESIGN AND/OR OPERATIONAL VARIABLE(S) OF INTEREST AND NOTE LINE NUMBER(S) (21000 TO 25990).
2. TRACE SELECTED LINE(S) ACROSS APPROPRIATE TABLES AND NOTE WHICH REPORTS (COLUMNS) CONTAIN TYPES OF DATA (T,Q,U, ETC.) OF INTEREST.
3. SEE LAST PAGES OF TABLE 1 FOR DESCRIPTIONS OF NOTED ITEMS AND X'S,
4. SEE BIBLIOGRAPHY FOR FULL TITLES OF REPORTS.
5. SEE WES MP HL ______ FOR DATA LOCATIONS WITHIN REPORTS.

DESIGN AND OPERATIONAL VARIABLES

(1) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A01 Possum Kingdom Spillway TM 111-1	A02 Possum Kingdom Submerged Bucket TM 111-2	A03 Possum Kingdon Baffles TM 111-3	A04 St. Lucie Canal L&D TM 153-1	A05 Santee River Spillway TM 168-1	A06 Canton Spillway TM 190-1	A07 Bluestone Baffles TM 2-243	A08 Demopolis L&D TM 2-252	A09 Morganza Floodwy Contr Structure TM 2-326	A10 Jim Woodruff L&D TM 2-340	A11 Cheatham Emergency Dam TM 2-358	A12 Cheatham Spillway Gate TM 2-381	A13 New Cumberland L&D TM 2-386	A14 Gavin's Point Spillway TM 2-404	A15 Old River L Sill Multi-Leaf Gates TR 2-447 Rpt 1	A16 Old River L Sill Dnstr Ch Riprap TR 2-447 Rpt 2	A17 Old River L Sill Control Struct TR 2-447 Rpt 3	A18 Warrior Spillway TR 2-485	A19 Old River O'bank Panel Gates TR 2-491	A20 Old River Closure Dam TR 2-496
21000 UPSTREAM APPROACH																				
21100 Channel				VC WE		QY				UVC WYI		QYL		QVCB WYI			UC BW			
21110 Direction						QUV CBY										UVB WER	VC			
21120 Shape						UVC BY		VC		VC										
21121 Invert El																				
21122 Width																				
21123 Side Slopes																				
21124 Bottom Slope																				
21130 Dikes										VC				VC BW						
21140 Noted Items																				
21200 Training Walls				C		QUV CBY			QVCB WYEZ					CB WY			UCB WY	C		
21300 Guide/Guard Walls										VC WH										
21400 Riprap																				
21410 Bottom																				
21420 Side																				
21430 Size																				R
21440 Thickness																				
21450 Slope																				R
21460 Noted Items																				
21500 Noted Items																				
21900 Operation																				
21910 Pool El				C		VCY		VC		VCN		QYL		VC BW			UCB WY	C		
21920 TW El				C						VCN							UCB WY			
21930 Type Flow																				
21931 Free/Submerged																				
21932 Gated/Uncontrolled																				
21933 Unit Discharge				C		VCY								VC BW						R
21940 Gate Schedule										VC										
21941 Single																	UC			
21942 Multiple																	UC BW			
21943 Locations																	UC			
21950 Gate Opening										VCN										
21951 Uniform				C																
21952 Variable																				
21960 Gate Submergence																				
21970 Gate Speed																				
21980 Other Factors																				
21981 Ice/Debris																				
21982 Loose Barges																				
21983 Waves																				
21984 Power Discharge																				
21990 Noted Items																				

(2)	A21 W F George L&D TR 2-519	A22 Jackson L&D TR 2-531	A23 Dardanelle L&D TR 2-558	A24 Markland Gates & Spillway TR 2-566	A25 Greenup Gates & Spillway TR 2-572	A26 Columbia Gates & Spillway TR 2-578	A27 Maxwell/Opekiska Gates & Basins TR 2-579	A28 New Cumberland Gates & Basin TR 2-585	A29 Pike Island Stilling Basin TR 2-586	A30 C&S Florida Proj Spillway TR 2-633	A31 Millers Ferry Gates & Basin TR 2-643	A32 Proctor Spillway TR 2-645	A33 Arkansas R Dams Overflow Embank TR 2-650	A34 Arkansas R Dams Lo-Head Spillway TR 2-655 & App A	A35 Oahe Spillway TR 2-657	A36 Belleville Stilling Basin TR 2-687	A37 Barkley Spillway TR 2-689	A38 Cannelton Spillway & Gates TR 2-710	A39 Hannibal Spillway TR 2-731	A40 Holt L&D TR 2-745	A41 Hugo Spillway TR H-69-15	A42 Copan Spillway TR H-70-09	A43 Oakley Spillway TR H-70-13 & App	A44 Arkansas R Dams Gate Vibration TR H-71-05	A45 Miss R L&D 15 Spillway & Gates TR H-73-15
21000	VC																								
21100	VC WY		VC			V		C							VCW YI		VCN			UVCB WYN	VC WS		VCW		
21110																									
21120																									
21121	C		QY							QY		QYZ	E								VC WS				
21122																									
21123																									
21124																									
21130																					CW				
21140																	VC				VC WS		QUVC BWYR		
21200	VC WY																						VC BW		
21300			VC														CN			Y					
21400																									
21410													QE RL												
21420																							R		
21430													QE RL												
21440													QE RL												
21450																									
21460																									
21500			X																						
21900																					CW				
21910	VC												QE RL				VC			VCN	VC		UVC BWR		
21920	VC												QE RL								VC				
21930																									
21931													QRL												
21932																									
21933													QE RL												
21940																									
21941																									
21942			VCX																						
21943			VCX																						
21950																				VCN			UVC BWR		
21951	VCY																								
21952			X																						
21960																									
21970																									
21980																									
21981			X																						
21982																									
21983																									
21984	VCY																								
21990																									

(3) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A46 Aliceville Spillway TR H-74-10	A47 Columbus Spillway TR H-74-13	A48 Old River L Sill Proto Vibration TR H-76-15	A49 Old River L Sill Operation TR H-77-02	A50 Ozark Proto Spillway TR H-77-06	A51 Red River L&D 1 Spillway TR H-77-13	A52 Tenn-Tom Canal Spillways A&B TR H-78-21	A53 Barkley Proto Gate Vibr TR HL-79-08	A54 Grays Landing Spillway TR HL-81-13	A55 Barkley Gate & Bulkhead TR HL-83-12	A56 Baffle Piers Cavitation MP 2-154	A57 Navigation Dam Swellhead MP 2-468	A58 Spillway Gates Vibration MP 2-484	A59 Overflow Embank Riprap MP 2-552	A60 Vert Lift Gates Discharge MP 2-606	A61 Spwy Tow Curves Pressures MP 2-625	A62 Sbmrgd Rock Weir Propeller Wash MP 2-821	A63 Rend L Sub Dams Riprap Cover MP H-68-01	A64 Baffled Basins Design Trends MP H-69-01	A65 Baffle Piers Drag Forces MP H-70-04
21000 UPSTREAM APPROACH																				
21100 Channel			UCB WY	VC		VCY	UC BW			CBW										
21110 Direction																				
21120 Shape	VC											QYL								
21121 Invert El			E			QY			QYZ											
21122 Width																				
21123 Side Slopes																				
21124 Bottom Slope																				
21130 Dikes																				
21140 Noted Items						VCY														
21200 Training Walls			UCB WYA																	
21300 Guide/Guard Walls																				
21400 Riprap																				
21410 Bottom						R														
21420 Side																				
21430 Size				R													R	R		
21440 Thickness																		R		
21450 Slope																				
21460 Noted Items																				
21500 Noted Items																				
21900 Operation																				
21910 Pool El			A	V			UC BW													
21920 TW El			A	V		R														
21930 Type Flow																				
21931 Free/Submerged				V																
21932 Gated/Uncontrolled						VR	UC BW													
21933 Unit Discharge																				
21940 Gate Schedule																				
21941 Single																				
21942 Multiple																				
21943 Locations																				
21950 Gate Opening				V		VR	UC BW													
21951 Uniform																				
21952 Variable																				
21960 Gate Submergence			A																	
21970 Gate Speed																				
21980 Other Factors																				
21981 Ice/Debris																				
21982 Loose Barges																				
21983 Waves																				
21984 Power Discharge																				
21990 Noted Items																				

4	A66	A67	A68	A69	A70	A71	A72	A73	A74	A75	A76	A77	A78	A79	A80	A81	A82	A83	A84	A85	A86	A87	A88	A89	A90
	Spillway Exit Ch Bottom Shape MP H-71-05	Tainter Gates CE Project Data MP H-72-07	Spillway Nappe Upper Surface MP H-73-04	Spillway Crest Design Profile MP H-73-05	Spillway Chute Flow Surface MP H-76-19	Old River O'bank Outlet Channel MP HL-80-05	Low-Ogee Crest Pressure Fluct CR H-71-01	Ohio R L&D 37 Spillway Paper D	RR Embankment O'flow Erosion Paper R	St. Lucie Canal Spillways Paper 14	St. Clair River Submerged Sills Paper 16	Hastings Spillway STP 01	Sand Dams O'flow Discharge STP 02	Kiskiminitas 2 Spillway & Chnnl STP 03	Beartrap Dam Chamber Silting STP 04	Marmet L&D STP 05	Miss R L&D 15 Dam & Spillway STP 07	App A to STP 07 Model vs Proto STP 09	Monongahela 4 Crest & Basin STP 12	Roller Gates Discharge Coef STP 13	Montgomery Is Channel & Spwy STP 14	Winfield Channel & Spwy STP 15	Miss R 5-5A-8 Roller Gt Coef STP 17	Miss R L&D 26 Chnl & Cofferdam STP 20	Peoria/La Grange Wicket Discharge STP 23
21000																									
21100														QVC YIL		QVC YL	QVCY NLX	QV YL		UC BW	QVCY IENL	QVCY INL		VCI	C
21110																	QYL								
21120																									
21121		X	Q	QY										QYL					QUW YZX		QCY ENL		QVY	E	
21122																					C				
21123																									
21124																									
21130																					VCI EN	QVCY INL			
21140																	VC								
21200					C																C			VCE	
21300														VCI		QVC YL	CNX				QVCY INL	QVCY INL			
21400																									
21410									TYR																
21420																									
21430																									
21440																									
21450																									
21460																									
21500																	QVCY NLX								
21900																									
21910														QVY IL		QVC YL	QCY NLX	QV YL			QVCY IENL	QVCY INL		VC IE	
21920																QYL		QV YL			QVCY IENL	QVCY INL		VCI	
21930																									
21931																QYL		VY			QYL				
21932																									
21933																					QYL				
21940																		YL			QC YL				
21941																QYL									
21942																QV YL	QCY NLX				QYL				
21943																QV YL	QCY NX				QC YL				
21950														VC YI		C		YL			QVCY IENL	QVCY INL			
21951																									
21952																									
21960																									
21970																									
21980																									
21981																	CNX								
21982																									
21983																									
21984																						VC IN			
21990																	QYL	VYL				QVCY INL			

(5) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A91 Miss R L&D 20 Stilling Basin STP 24	A92 Miss R L&D 7 Spwy Culverts STP 29	A93 Chanoine Wickets Discharge Coef STP 30	A94 Miss R 5-5A-8 Tainter Gt Coef STP 31	A95 Miss R L&D 22 S..lling Basin STP 33	A96 Roller Gate Stilling Basins STP 36	A97 Sub Tainter Gate Coef & Basin STP 37	A98 App A to STP 13 Roller Gate Coef STP 39	A99 Miss R L&D 4 Gates & Basin STP 53	B01 Miss R L&D 1 Spillway Apron STP 63	B02 SAF Lower L&D Tntr Gt & Basin STP 69	B03 Roller Gates Pressures STP 77	B04 Bonneville Spillway Press BHL 3-1	B05 McNary Cffrdms & T-race BHL 20-1	B06 McNary Spillway & Gates BHL 21-1	B07 Ice Harbor Cffrdms & T-race BHL 22-1	B08 Ice Harbor Spillway BHL 31-1	B09 The Dalles Spillway & Basin BHL 55-1	B10 Bonneville Rev Still Basin BHL 65-1	B11 John Day Spwy & Dvrsn Gap BHL 97-1
21000 UPSTREAM APPROACH																				
21100 Channel			C			VC			QVC YIL	QY				DVC YN	W	QVC YI				UE
21110 Direction																				
21120 Shape																VY				
21121 Invert El							QYL									VC YI		QUB WYZ		
21122 Width																				
21123 Side Slopes																				
21124 Bottom Slope																				
21130 Dikes																VC YI				
21140 Noted Items																				
21200 Training Walls									E											
21300 Guide/Guard Walls														DVC N						
21400 Riprap																				
21410 Bottom																				
21420 Side																				
21430 Size																				
21440 Thickness																				
21450 Slope																				
21460 Noted Items																				
21500 Noted Items																				
21900 Operation																				
21910 Pool El									QC YL					VCY		QVC YI		YZ		
21920 TW El									QC YL					VCY		VC YI				
21930 Type Flow																				
21931 Free/Submerged																				
21932 Gated/Uncontrolled																		Z		
21933 Unit Discharge																				
21940 Gate Schedule									QC YL							VC YI		Z		
21941 Single																				
21942 Multiple														DV CN						
21943 Locations																				
21950 Gate Opening						VC			QC YL							VC YI		Z		
21951 Uniform																				
21952 Variable																				
21960 Gate Submergence						VC														
21970 Gate Speed																				
21980 Other Factors																				
21981 Ice/Debris						V														
21982 Loose Barges														DV CN						
21983 Waves																				
21984 Power Discharge														DV CN		VC YI				
21990 Noted Items																				

8	B12	B13	B14	B15	B16	B17	B18	B19	B20	B21
	Lower Monumental Spillway & Basin BHL 104-1	Little Goose Spillway & Basin BHL 114-1	Bonneville Gate Lip Mod BHL 136-1	Allegheny L&D 9 Spwy & Channel CIT A4L9	Wicket Gate Operating Forces CIT C3	Bluestone Baffle Pier Cav CIT B57R	Bonneville Baffle Pier Cav CIT B6C	Tuscaloosa Stilling Basin CIT T8	Gallipolis Channel & Spwy CIT G35F	Gallipolis Proto Currents HDG35R
21000										
21100	CW	VCW YI		UDVC BWN					DV CN	VC NX
21110										
21120		VC WI								
21121										
21122										
21123										
21124										
21130				DV CN					DV CN	
21140									DV CN	
21200										
21300				DCN					DV CN	VC NX
21400										
21410										
21420										
21430										
21440										
21450										
21460										
21500		QVC WYI								
21900										
21910				VCN					DV CN	VC NX
21920				CN						VC NX
21930										
21931										
21932										
21933										
21940		VC WI								VC NX
21941										
21942										
21943										
21950									DV CN	VC NX
21951										
21952										
21960										
21970										
21980										
21981										
21982										
21983										
21984										VC NX
21990										

(7) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A01 Possum Kingdom Spillway TM 111-1	A02 Possum Kingdom Submerged Bucket TM 111-2	A03 Possum Kingdom Baffles TM 111-3	A04 St. Lucie Canal L&D TM 153-1	A05 Santee River Spillway TM 168-1	A06 Canton Spillway TM 190-1	A07 Bluestone Baffles TM 2-243	A08 Demopolis L&D TM 2-252	A09 Morganza Floodwy Contr Structure TM 2-326	A10 Jim Woodruff L&D TM 2-340	A11 Cheatham Emergency Dam TM 2-358	A12 Cheatham Spillway Gate TM 2-381	A13 New Cumberland L&D TM 2-386	A14 Gavin's Point Spillway TM 2-404	A15 Old River L Sill Multi-Leaf Gates TR 2-447 Rpt 1	A16 Old River L Sill Dnstr Ch Riprap TR 2-447 Rpt 2	A17 Old River L Sill Control Struct TR 2-447 Rpt 3	A18 Warrior Spillway TR 2-485	A19 Old River O'bank Panel Gates TR 2-491	A20 Old River Closure Dam TR 2-496
22000 CONTROL SILL	Q			Q		QV CY		QUVC BWYI	CBW	QU YX			QUV WY	QYP			QUB WY	UW YI		
22100 Crest El					YZ							QYL								
22200 Sill Shape									VY EZ	QUVB WYIZ								QY		
22210 Upstream Face						VY IZ														
22211 Shape																				
22212 Slope																				
22220 Top Width									QCB WY											
22230 Downstream Face																				UV ER
22231 Shape				VC YE																
22232 Slope						QVY IZ														
22233 Chute						QUVB WYIZ								UB WY						
22240 Noted Items		UB WE				QUB WYZ				QUVB WYI										UVY ERX
22300 Net Length																				
22400 Gate Bays					Y															
22410 Number																				
22420 Width																				
22500 Piers									YZ	QY				QYP			Y			
22510 Width																				
22520 Height																				
22530 Upstream Length																				
22540 Nose Shape						WY											UC BW			
22600 Navigable Pass								R												
22700 Cofferdams										QV CY										
22800 Noted Items				VCW		W														UYE RX
22900 Operation																				
22910 Pool El	Q	UB WE			YZ	QVY IZ		QVC	QYZ	QUVCB WYIZ		QYL	QY	QYP			QY	QLW YI		UV YX
22920 TW El		UB WE			YZ	VY IZ		QVC	QYZ	QUVCB WYIZX		QYL	QY	Y			QY	QUW YI		UV YX
22930 Type Flow										QUX										
22931 Free/Submerged					Y			QVC	Z	QUVB WYIZ			QY				QY	QUW YI		UER
22932 Gated/Uncontrolled										QY								UW YI		
22933 Unit Discharge		UB WE			YZ	VY IZ		VC	QYZ	QUVW YIZ			QY	YP						UV
22940 Gate Schedule										QUX							Y			
22941 Single						QYZ								P						
22942 Multiple					Y									P						
22943 Locations																				
22950 Gate Opening										QUX				P						
22951 Uniform																				
22952 Variable																				
22960 Gate Submergence																				
22970 Gate Speed																				
22980 Other Factors																				
22981 Ice/Debris																				
22982 Loose Barges																				
22983 Waves																				
22984 Power Discharge																				
22990 Noted Items				VCW		Z														R

8	A21 W F George L&D TR 2-519	A22 Jackson L&D TR 2-531	A23 Dardanelle L&D TR 2-558	A24 Markland Gates & Spillway TR 2-566	A25 Greenup Gates & Spillway TR 2-572	A26 Columbia Gates & Spillway TR 2-578	A27 Maxwell/Opekiska Gates & Basins TR 2-579	A28 New Cumberland Gates & Basin TR 2-585	A29 Pike Island Stilling Basin TR 2-586	A30 C&S Florida Proj Spillway TR 2-633	A31 Millers Ferry Gates & Basin TR 2-643	A32 Proctor Spillway TR 2-645	A33 Arkansas R Dams Overflow Embank TR 2-650	A34 Arkansas R Dams Lo-Head Spillway TR 2-655 & App A	A35 Oahe Spillway TR 2-657	A36 Belleville Stilling Basin TR 2-687	A37 Barkley Spillway TR 2-689	A38 Cannelton Spillway & Gates TR 2-710	A39 Hannibal Spillway TR 2-731	A40 Holt L&D TR 2-745	A41 Hugo Spillway TR H-69-15	A42 Copan Spillway TR H-70-09	A43 Oakley Spillway TR H-70-13 & App	A44 Arkansas R Dams Gate Vibration TR H-71-05	A45 Miss R L&D 15 Spillway & Gates TR H-73-15
22000	VC	QUV WYI	QB WY	UBW	UBW					QUB WY		QUB WYZ	WYZ	QUW YZL	QY			QY	QUB WY	UC BW					
22100													QUVW YERL	UBW									QUV CY		QWY
22200	QUVCW YIEZ		QUVW YIZ	QYL	QY	QUVBW YIZP	QY	UVB WY	QY				QUVW YERL							QY		QY	QY		
22210													ER							Z					
22211																									
22212												QYZ													
22220													QE RL								QUY ZP				
22230				UVZ PAX									UBW YRZ					QY		Z					
22231						UVBW YIER			UV BW		QUV BWY	QYZ		QUVBW YERZL											
22232		US																							
22233																									
22240	QV CY	US		Z									QUVBW YERL							QY					
22300																									
22400																									
22410																									
22420																									
22500			QUVW YIZ									QYZ								QY	QUY ZP		QY		WY
22510	QY																								
22520																									
22530																				CW					
22540			QB WY											Q											
22600																									
22700	QV CY																			UVC BWY					
22800			VC																						
22900																									
22910	QUVCW YIEZ	QY	QUVW YIZ	QY ZL		QV YR				QY	QU VY	QYZ	QUVBW YERZL	QUW YZL	QY			QY	QY	QVC YZ	QUY ZP	QY	QY		QWY
22920	QUVCW YIEZ	QY	QUVW YIZ	QY ZL		QY ZP	QY		QY	QY	QUV BWY		QUVBW YERZL	QUW YZL	QY			QY	QY	Z	QUY ZP				
22930																									
22931	QY	QY	QUVW YIZ	QY ZL		QY ZP	QY		QY	QY	QUV BWY		QUVBW YERZL	QUY ZL	QY			QY	QY	Z	QUV				QY
22932		QY	UVW YIZ			QY				QY	QU VY			QUW YZ	QY			QY	QY		QUY				
22933	UVWY IEZ		QY			QY							QUBWY ERZL	QYL											
22940																									
22941																									
22942																									
22943																									
22950				Z		QY ZP	QY		QY	QY	QU VY	Z		QUW YZ	QY			QY		Z					
22951	QYZ	QY																							
22952																									
22960				Z																					
22970																									
22980																									
22981																									
22982																									
22983																									
22984																									

(9) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A46 Aliceville Spillway TR H-74-10	A47 Columbus Spillway TR H-74-13	A48 Old River L Sill Proto Vibration TR H-76-15	A49 Old River L Sill Operation TR H-77-02	A50 Ozark Proto Spillway TR H-77-06	A51 Red River L&D 1 Spillway TR H-77-13	A52 Tenn-Tom Canal Spillways A&B TR H-78-21	A53 Barkley Proto Gate Vibr TR HL-79-08	A54 Grays Landing Spillway TR HL-81-13	A55 Barkley Gate & Bulkhead TR HL-83-12	A56 Baffle Piers Cavitation MP 2-154	A57 Navigation Dam Swellhead MP 2-468	A58 Spillway Gates Vibration MP 2-484	A59 Overflow Embank Riprap MP 2-552	A60 Vert Lift Gates Discharge MP 2-606	A61 Spwy Tow Curves Pressures MP 2-625	A62 Sbmrgd Rock Weir Propeller Wash MP 2-821	A63 Rend L Sub Dams Riprap Cover MP H-68-01	A64 Baffled Basins Design Trends MP H-69-01	A65 Baffle Piers Drag Forces MP H-70-04
22000 CONTROL SILL				QYZ	QVCYI EZPA	QY						QYL						QY		
22100 Crest El			F	QUVW YZLF		UVC BW								WYR					QV YX	
22200 Sill Shape				QUVW YZLF			QUVCB WYIZ		QUBW YZX					WYR	QY		QYI			
22210 Upstream Face																				
22211 Shape																				
22212 Slope							Z													
22220 Top Width																				
22230 Downstream Face													ZPA	UB WR						
22231 Shape																				
22232 Slope																				
22233 Chute																				
22240 Noted Items														UBW YR		Z	QY IR	UBW IR		
22300 Net Length							QY		QYZ											
22400 Gate Bays							QUVC BWY					QYL								
22410 Number																				
22420 Width																				
22500 Piers			A		CY PA		QUVCB WYIZ			SF		QYL								
22510 Width																				
22520 Height																				
22530 Upstream Length																				
22540 Nose Shape																				
22600 Navigable Pass																				
22700 Cofferdams																				
22800 Noted Items			A									QYL					QY IR			
22900 Operation																				
22910 Pool El			A	QY ZL		QY	QUC BWY		QUBW YZX			QYL		UBW YR		Z	QY	QY		
22920 TW El			A	QY ZL	VIZ PA	QY			QYZ			QYL		UBW YR			QY	R		
22930 Type Flow																				
22931 Free/Submerged				QY ZL	VIZ PA	QY			QUBW YZX					UBW YR						
22932 Gated/Uncontrolled				Z	VIZ PA	QY	QUC BWY													
22933 Unit Discharge														UBW YR		Z	QY	R		
22940 Gate Schedule																				
22941 Single					VIZ PA															
22942 Multiple				QY																
22943 Locations																				
22950 Gate Opening			A	QYZ	VIZ PA		UCB WY													
22951 Uniform																				
22952 Variable																				
22960 Gate Submergence			A																	
22970 Gate Speed																				
22980 Other Factors																				
22981 Ice/Debris																		R		
22982 Loose Barges																				
22983 Waves																				
22984 Power Discharge																				
22990 Noted Items												QYL					VIR			

(10)	A66 Spillway Exit Ch Bottom Shape MP H-71-05	A67 Tainter Gates CE Project Data MP H-72-07	A68 Spillway Nappe Upper Surface MP H-73-04	A69 Spillway Crest Design Profile MP H-73-05	A70 Spillway Chute Flow Surface MP H-76-19	A71 Old River O'bank Outlet Channel MP HL-80-05	A72 Low-Ogee Crest Pressure Fluct CR H-71-01	A73 Ohio R L&D 37 Spillway Paper D	A74 RR Embankment O'flow Erosion Paper R	A75 St. Lucie Canal Spillways Paper 14	A76 St. Clair River Submerged Sills Paper 16	A77 Hastings Spillway STP 01	A78 Sand Dams O'flow Discharge STP 02	A79 Kiskiminitas 2 Spillway & Chnnl STP 03	A80 Beartrap Dam Chamber Silting STP 04	A81 Marmet L&D STP 05	A82 Miss R L&D 15 Dam & Spillway STP 07	A83 App A to STP 07 Model vs Proto STP 09	A84 Monongahela 4 Crest & Basin STP 12	A85 Roller Gates Discharge Coef STP 13	A86 Montgomery Is Channel & Spwy STP 14	A87 Winfield Channel & Spwy STP 15	A88 Miss R 5-5A-8 Roller Gt Coef STP 17	A89 Miss R L&D 26 Chnl & Cofferdam STP 20	A90 Peoria/La Grange Wicket Discharge STP 23
22000			QY		C		UBW YI		TUVB WYER		L			QYL		QV YL		YL		UC BW	QYL	QYL		VYI	
22100		X	Q	QY			QY				VYL												QVY		
22200	UB WE	X	Y		QY		QY	QY		TUB WER	VY IL		QUVB WYLX						TQUVB WYIEZ LX						
22210		X																							
22211				QY												UBW YI									
22212			Q	Y	QY																				
22220																									
22230		X					UBWY IZP										UB WY					UB WZ		UVB WYI	
22231				Y												UBW YI						TUB WER			
22232																									
22233					WY																				
22240		X			WY	QY					VYL		QUB WYL						QYL						
22300					QY			QY																	
22400																									
22410								QY																	
22420																									
22500														QYL		QU YL				UC BW					
22510																									
22520																									
22530																									
22540																									
22600											VY IL					QUY NL								VC IN	QY
22700																QVC YNL	QCY	QV YL				QVCY INL		TVCY IEL	
22800	TUB WE													VI	VYI EZ	QVC YNL					QCY NL				QUVB WYIE
22900																									
22910			QY		QWY	QY	QUBW YIZP	QY	TUVB WYR		VYL		QUB WYL	QYL		QVC YNL			QUVW YZLX		QYL			VC YL	QV YI
22920							UBW YIP	QY	TUVB WYR		VYL		QUB WYL	QYL		QUVC BWY INL			QUVW YZLX		QYL	Z		VC YL	QV YI
22930																									
22931													QUB WYL			QYL			QUWY ZLX		QYL	Z			
22932																									
22933					WY	QY	QUBW YIZP				VYL		QUB WYL			UBWY			QUVW YZLX						
22940																									QV YI
22941																QYL									
22942																QVYL								CE	
22943																QVYL									
22950																QUB WYL	UB WY					Z			
22951																									
22952																									
22960																									
22970																									
22980																									
22981																									
22982																									
22983																									
22984																									
22990							P								YZ	QYL						Z		YL	

(11) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A91 Miss R L&D 20 Stilling Basin STP 24	A92 Miss R L&D 7 Spwy Culverts STP 29	A93 Chanoine Wickets Discharge Coef STP 30	A94 Miss R 5-5A-8 Tainter Gt Coef STP 31	A95 Miss R L&D 22 Stilling Basin STP 33	A96 Roller Gate Stilling Basins STP 36	A97 Sub Tainter Gate Coef & Basin STP 37	A98 App A to STP 13 Roller Gate Coef STP 39	A99 Miss R L&D 4 Gates & Basin STP 53	B01 Miss R L&D 1 Spillway Apron STP 63	B02 SAF Lower L&D Tntr Gt & Basin STP 69	B03 Roller Gates Pressures STP 77	B04 Bonneville Spillway Press BHL 3-1	B05 McNary Cffrdms & T-race BHL 20-1	B06 McNary Spillway & Gates BHL 21-1	B07 Ice Harbor Cffrdms & T-race BHL 22-1	B08 Ice Harbor Spillway BHL 31-1	B09 The Dalles Spillway & Basin BHL 55-1	B10 Bonneville Rev Still Basin BHL 65-1	B11 John Day Spwy & Dvrsn Gap BHL 97-1
22000 CONTROL SILL	QYL	QY				QYZ	Z	YLX	QYL	QY	UWY				CB WY		QY			QYZ
22100 Crest El							QYL		QC YL					VY						UVBW YIE
22200 Sill Shape							QYL				YZ									
22210 Upstream Face																	YZ			
22211 Shape															QY ZP			QUB WYZ		
22212 Slope																				
22220 Top Width													YZ				YZ			
22230 Downstream Face						TE					ZX	ZX			Y		UBW YZ			UBW YIE
22231 Shape					TUVB WIE								YZX		QY ZP			QUB WYZ		UVB WYI
22232 Slope																				
22233 Chute																				
22240 Noted Items																				UVBW YIE
22300 Net Length									QVC YEL											
22400 Gate Bays															CB WY		UB WY			UVC WIE
22410 Number																				
22420 Width								QY	QY						QY				QY	
22500 Piers				QYL		V									QCBW YZP		QY	YZ		CW
22510 Width																				
22520 Height																				
22530 Upstream Length				QY																QYZ
22540 Nose Shape							QYL										UBW YZ			QYZ
22600 Navigable Pass																				
22700 Cofferdams														VCY		QVCW YIE				
22800 Noted Items		QUB WYE				E			QVC YIL						Z					QYZ
22900 Operation																				
22910 Pool El	QYL	QY		QYL		Z	Z		QYL	QY	UWY	ZX		VY	QCB WYZ	VCW YI	QUB WYZ	QYZ		QUVY IEZ
22920 TW El	QYL	QY		QYL		QYZ	Z		QYL		UWY ZX	ZX		VY	Z	VCW YI		YZ		UV IE
22930 Type Flow						Z														
22931 Free/Submerged		QY				Z	Z													
22932 Gated/Uncontrolled															CBW YZ		Z	QYZ		
22933 Unit Discharge																				
22940 Gate Schedule															CBW YZ	VC YI	YZ	Z		YZ
22941 Single																				
22942 Multiple																				
22943 Locations																				
22950 Gate Opening						VZ	Z				UWY ZX	ZX	YZ		CBW YZ		Z	Z		QYZ
22951 Uniform																				
22952 Variable																				
22960 Gate Submergence						VZ	Z				UW YZ	Z								
22970 Gate Speed																				
22980 Other Factors																				
22981 Ice/Debris						V														
22982 Loose Barges																				
22983 Waves																				
22984 Power Discharge																VC YI				
22990 Noted Items													YZX		CBW YZ					

12	B12 Lower Monumental Spillway & Basin BHL 104-1	B13 Little Goose Spillway & Basin BHL 114-1	B14 Bonneville Gate Lip Mod BHL 136-1	B15 Allegheny L&D 9 Spwy & Channel CIT A4L9	B16 Wicket Gate Operating Forces CIT C3	B17 Bluestone Baffle Pier Cav CIT B57R	B18 Bonneville Baffle Pier Cav CIT B6C	B19 Tuscaloosa Stilling Basin CIT T8	B20 Gallipolis Channel & Spwy CIT G35F	B21 Gallipolis Proto Currents HDG35R
22000	QY		QY	UVC BW			QY	V	QY IZ	
22100	QY									
22200	CW YZ	QCW YZ								
22210										
22211										
22212										
22220										
22230						VY	UVC BYX			
22231										
22232										
22233										
22240										
22300										
22400	CW	CWY	CB WY				QUC BY		UBW IZ	
22410										
22420										
22500	QCW YZ	QCW YZ	Y							
22510										
22520										
22530										
22540										
22600					UBW					
22700	Z									
22800	QCY		QCB WY	UVC BW			UC BY			
22900										
22910	QYZ	QCW YZ	QY	UVC BW		VY	QVY	V	QY	
22920			QY	UVC BW					QUBW YIZ	
22930										
22931									QUBW YIZ	
22932		QC WY								
22933										
22940	CW	QYZ								
22941										
22942										
22943										
22950	CWZ	QCW YZ					QUV CBY		QUBW YIZ	
22951										
22952										
22960										
22970										
22980										
22981										
22982										
22983										
22984										
22990										

(13)

DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A01 Possum Kingdom Spillway TM 111-1	A02 Possum Kingdom Submerged Bucket TM 111-2	A03 Possum Kingdon Baffles TM 111-3	A04 St. Lucie Canal L&D TM 153-1	A05 Santee River Spillway TM 168-1	A06 Canton Spillway TM 190-1	A07 Bluestone Baffles TM 2-243	A08 Demopolis L&D TM 2-252	A09 Morganza Floodwy Contr Structure TM 2-326	A10 Jim Woodruff L&D TM 2-340	A11 Cheatham Emergency Dam TM 2-358	A12 Cheatham Spillway Gate TM 2-381	A13 New Cumberland L&D TM 2-386	A14 Gavin's Point Spillway TM 2-404	A15 Old River L Sill Multi-Leaf Gates TR 2-447 Rpt 1	A16 Old River L Sill Dnstr Ch Riprap TR 2-447 Rpt 2	A17 Old River L Sill Control Struct TR 2-447 Rpt 3	A18 Warrior Spillway TR 2-485	A19 Old River O'bank Panel Gates TR 2-491	A20 Old River Closure Dam TR 2-496
23000 GATES AND BULKHEADS				Q						QUW YIX			QUV WY		UBW			QUVW YIF	UF	
23100 Type																				
23200 Shape											CB WF	QUBW EZF			I					
23210 Height															FA					
23220 Radius						QY														
23230 Tilt																				
23240 Lip						QY									FA					
23300 Location on Sill																				
23400 Weight																				
23500 Hoist												F			FA				F	
23600 Emergency Closure											CB WF				FA			VY		
23700 Noted Items				Q								FX			FA					
23900 Operation																				
23910 Pool El				Q		QY						F	QY		FA				F	
23920 TW El										QUW YIX	F	QE ZF	QY		FA			QUVW YIF	F	
23930 Type Flow										QUX										
23931 Free/Submerged									CBW	UW YI		QZ FX	QY		FA			QY	UF	
23932 Gated/Uncontrolled															FA					
23933 Unit Discharge										UW YI			QY							
23940 Gate Schedule										QUX										
23941 Single															FA				F	
23942 Multiple				Q											FA				F	
23943 Locations											F				FA				F	
23950 Gate Opening										QUW YIX		QEZ FX	QY		I			QUVW YIF		
23951 Uniform				Q		QY									FA					
23952 Variable																				
23960 Gate Submergence												QEZ FX			FA			VY		
23970 Gate Speed																				
23980 Other Factors																				
23981 Ice/Debris																				
23982 Loose Barges																				
23983 Waves																				
23984 Power Discharge																				
23990 Noted Items																				

(14)	A21	A22	A23	A24	A25	A26	A27	A28	A29	A30	A31	A32	A33	A34	A35	A36	A37	A38	A39	A40	A41	A42	A43	A44	A45
	W F George L&D TR 2-519	Jackson L&D TR 2-531	Dardanelle L&D TR 2-558	Markland Gates & Spillway TR 2-566	Greenup Gates & Spillway TR 2-572	Columbia Gates & Spillway TR 2-578	Maxwell/Opekiska Gates & Basins TR 2-579	New Cumberland Gates & Basin TR 2-585	Pike Island Stilling Basin TR 2-586	C&S Florida Proj Spillway TR 2-633	Millers Ferry Gates & Basin TR 2-643	Proctor Spillway TR 2-645	Arkansas R Dams Overflow Embank TR 2-650	Arkansas R Dams Lo-Head Spillway TR 2-655 & App A	Oahe Spillway TR 2-657	Belleville Stilling Basin TR 2-687	Barkley Spillway TR 2-689	Cannelton Spillway & Gates TR 2-710	Hannibal Spillway TR 2-731	Holt L&D TR 2-745	Hugo Spillway TR H-69-15	Copan Spillway TR H-70-09	Oakley Spillway TR H-70-13 & App	Arkansas R Dams Gate Vibration TR H-71-05	Miss R L&D 15 Spillway & Gates TR H-73-15
23000	QYZ	QVY	QUVW YIZ	UBW	UBW	QY					QU VY			QUY	QU VY				QUB WY	Z	QUY		QY	A	QYX
23100				QUVYZ PFAX	QY FA			QUV YFA	QY	QUB WY								UB WX							
23200				F			QY											UV							
23210																									
23220																		QUY					QY		QUW YFA
23230																									
23240				ZP AX	F																			A	
23300																									
23400																									
23500				FA	FA			FA																	
23600					FA			CF																	
23700				AX	F													QU VY							UW FA
23900																									
23910	QYZ	QY	UVW YI	ZFA	FA	QY		FA		QUB WY	QY			QUY					QY		QUY		QY	A	QY
23920		QY	QUV WYI	QYZ FA	QY FA	QY	QY	QY FA	QY	QUB WY	QY			QUY				QY	QY		QUY		VC	A	UW FAX
23930																									
23931		QVY	QUV WYI	QYZ PFA		QY	QY	QY	QY	QUB WY	QY			QUY				QY	QY		QUY				
23932		QY	UVW YI	QY						QUB WY	QY							QY	QY						
23933	Z		QY										VW YR												
23940		V																							
23941																									
23942					F			F																	
23943								F																	
23950		V		QYZP FAX	QY FA	QYX	QY	QCY FA	QY	QUB WY	QY			QUY				QY			QUY		QY	A	QUWY FAX
23951	QYZ	QY																							
23952																									
23960				QYZP FAX	QY FA			QCY FA																	
23970																									
23980																									
23981						X																			X
23982																									
23983																									
23984																									
23990					F																				

(15) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A46 Aliceville Spillway TR H-74-10	A47 Columbus Spillway TR H-74-13	A48 Old River L Sill Proto Vibration TR H-76-15	A49 Old River L Sill Operation TR H-77-02	A50 Ozark Proto Spillway TR H-77-06	A51 Red River L&D 1 Spillway TR H-77-13	A52 Tenn-Tom Canal Spillways A&B TR H-78-21	A53 Barkley Proto Gate Vibr TR HL-79-08	A54 Grays Landing Spillway TR HL-81-13	A55 Barkley Gate & Bulkhead TR HL-83-12	A56 Baffle Piers Cavitation MP 2-154	A57 Navigation Dam Swellhead MP 2-468	A58 Spillway Gates Vibration MP 2-484	A59 Overflow Embank Riprap MP 2-552	A60 Vert Lift Gates Discharge MP 2-606	A61 Spwy Tow Curves Pressures MP 2-625	A62 Sbmrgd Rock Weir Propeller Wash MP 2-821	A63 Rend L Sub Dams Riprap Cover MP H-68-01	A64 Baffled Basins Design Trends MP H-69-01	A65 Baffle Piers Drag Forces MP H-70-04
23000 GATES AND BULKHEADS			FAX	QUW YZF	QPA	QY		PA		SFA			A							
23100 Type																				
23200 Shape							QUC BWY						ZPA							
23210 Height						V	UBW													
23220 Radius																				
23230 Tilt																				
23240 Lip								PA		SF					Q					
23300 Location on Sill																QY				
23400 Weight																				
23500 Hoist			FX	F						SFA										
23600 Emergency Closure										CBW FA			A							
23700 Noted Items			FX	QUV WYZ						SF			IA							
23900 Operation																				
23910 Pool El			FA	QYF		QY	QUC BWY			CBW FA			A		QY					
23920 TW El			FA	QYF		QY	UC BW			CBW SFA			A							
23930 Type Flow																				
23931 Free/Submerged				QYF	Q	QY				CBW SFA										
23932 Gated/Uncontrolled				QY	Q		UC BW								Y					
23933 Unit Discharge																				
23940 Gate Schedule																				
23941 Single					QCY															
23942 Multiple																				
23943 Locations																				
23950 Gate Opening			FA	QYF	Q	QY	QUC BWY	PA		SFA			ZPA		QY					
23951 Uniform																				
23952 Variable																				
23960 Gate Submergence			FA	F				PA		CBW SFA			IZP AX							
23970 Gate Speed										FA										
23980 Other Factors																				
23981 Ice/Debris																				
23982 Loose Barges																				
23983 Waves																				
23984 Power Discharge																				
23990 Noted Items										FA										

16	A66	A67	A68	A69	A70	A71	A72	A73	A74	A75	A76	A77	A78	A79	A80	A81	A82	A83	A84	A85	A86	A87	A88	A89	A90
	Spillway Exit Ch Bottom Shape MP H-71-05	Tainter Gates CE Project Data MP H-72-07	Spillway Nappe Upper Surface MP H-73-04	Spillway Crest Design Profile MP H-73-05	Spillway Chute Flow Surface MP H-76-19	Old River O'bank Outlet Channel MP HL-80-05	Low-Ogee Crest Pressure Fluct CR H-71-01	Ohio R L&D 37 Spillway Paper D	RR Embankment O'flow Erosion Paper R	St. Lucie Canal Spillways Paper 14	St. Clair River Submerged Sills Paper 16	Hastings Spillway STP 01	Sand Dams O'flow Discharge STP 02	Kiskiminitas 2 Spillway & Chnnl STP 03	Beartrap Dam Chamber Silting STP 04	Marmet L&D STP 05	Miss R L&D 15 Dam & Spillway STP 07	App A to STP 07 Model vs Proto STP 09	Monongahela 4 Crest & Basin STP 12	Roller Gates Discharge Coef STP 13	Montgomery Is Channel & Spwy STP 14	Winfield Channel & Spwy STP 15	Miss R 5-5A-8 Roller Gt Coef STP 17	Miss R L&D 26 Chnl & Cofferdam STP 20	Peoria/La Grange Wicket Discharge STP 23
23000												QY										QYL	QUV WYE	QY	QUVCB WYIE
23100								QY												QY					
23200		X																							
23210		X																							
23220		X																							
23230		X																							
23240																									
23300		X																							
23400																									
23500																									
23600		X																							
23700															TVY IE							E			QUVCB WYIE
23900																									
23910												QY								QY			QUY	QY	QUVB WYI
23920												QY								QY			QUY	QY	QUVB WYI
23930																									
23931												QY								QY			QUY	QY	
23932																									
23933							P																		
23940																				QY					QVY IE
23941																				QUC BWY					C
23942																				QY					UC BW
23943																				QY					UC BW
23950		X													TE					QUC BWY			QUY	QY	
23951																									
23952																									
23960																							QY		
23970																									
23980																									
23981																									
23982																									
23983																									
23984																									
23990															TE										

(17) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A91 Miss R L&D 20 Stilling Basin STP 24	A92 Miss R L&D 7 Spwy Culverts STP 29	A93 Chanoine Wickets Discharge Coef STP 30	A94 Miss R 5-SA-8 Tainter Gt Coef STP 31	A95 Miss R L&D 22 Stilling Basin STP 33	A96 Roller Gate Stilling Basins STP 36	A97 Sub Tainter Gate Coef & Basin STP 37	A98 App A to STP 13 Roller Gate Coef STP 39	A99 Miss R L&D 4 Gates & Basin STP 53	B01 Miss R L&D 1 Spillway Apron STP 63	B02 SAF Lower L&D Tntr Gt & Basin STP 69	B03 Roller Gates Pressures STP 77	B04 Bonneville Spillway Press BHL 3-1	B05 McNary Cffrdms & T-race BHL 20-1	B06 McNary Spillway & Gates BHL 21-1	B07 Ice Harbor Cffrdms & T-race BHL 22-1	B08 Ice Harbor Spillway BHL 31-1	B09 The Dalles Spillway & Basin BHL 55-1	B10 Bonneville Rev Still Basin BHL 65-1	B11 John Day Spwy & Dvrsn Gap BHL 97-1
23000 GATES AND BULKHEADS	QU YL		QUCB WYX	QV YL	UBWY ZLF	TQCY EZF	QYL	QUBW YLX	QYL		QUW YZ	ZFX		UZ	CBW YF		QY	QYX	QY	
23100 Type	UY								TQUVC BWYIE EZF			ZX			W					
23200 Shape						QYZ	TQUVB WYEZL F					ZFX								
23210 Height																				
23220 Radius					E		QY													
23230 Tilt							QYL													
23240 Lip															YI					
23300 Location on Sill													YZ		QZ PF					
23400 Weight																				
23500 Hoist					ZF	TE ZF	ZF		VZF						F					
23600 Emergency Closure																				
23700 Noted Items			QUB WYX				TUVY EZF					Z			QY					
23900 Operation																				
23910 Pool El	QU YL		QYX	QV YL	UY ZL	UYZ	QY ZL	QUBW YLX	Y		QU WY	ZX					QY	QY	QY	
23920 TW El	QUY		QYX	QV YL	UBW YZL	QYZ	QYZ LF	QUY LX	QVY ZF		UBW YZ	ZX								
23930 Type Flow																				
23931 Free/Submerged	QYL			QYL	UBW YZL	ZF	QYZ LF	U	QVY ZF											
23932 Gated/Uncontrolled																				
23933 Unit Discharge																				
23940 Gate Schedule															W		QY	X	QY	
23941 Single																				
23942 Multiple			QYX																	
23943 Locations																				
23950 Gate Opening	QU YL		QYX	QUVB WYL	ZF	QY ZF	QUY ZLF	QU YL	QVY ZF		QUW YZ	ZFX	YZ		QF		QY	QYX	QY	
23951 Uniform																				
23952 Variable																				
23960 Gate Submergence				QUB WY	ZF	QY ZF	QUY ZLF	QUY LX	Y		QUW YZ	Z								
23970 Gate Speed																				
23980 Other Factors																				
23981 Ice/Debris					ZF		ZF											X		
23982 Loose Barges																				
23983 Waves																				
23984 Power Discharge																				
23990 Noted Items					ZF		ZF						YZ		QWF					

18	B12 Lower Monumental Spillway & Basin BHL 104-1	B13 Little Goose Spillway & Basin BHL 114-1	B14 Bonneville Gate Lip Mod BHL 136-1	B15 Allegheny L&D 9 Spwy & Channel CIT A4L9	B16 Wicket Gate Operating Forces CIT C3	B17 Bluestone Baffle Pier Cav CIT B57R	B18 Bonneville Baffle Pier Cav CIT B6C	B19 Tuscaloosa Stilling Basin CIT T8	B20 Gallipolis Channel & Spwy CIT G35F	B21 Gallipolis Proto Currents HDG35R
23000	QYZ	QYZ					QY		IZ	
23100					UBW YF					
23200										
23210										
23220										
23230										
23240	QY		UBYIZ PFAX							
23300										
23400										
23500					F					
23600										
23700					WYF					
23900										
23910	QY	QY	ZF FA		UBW YF		QY			
23920			YIZ PFA		UBW YF				UBW IZ	
23930			CB WY							
23931									UBW IZ	
23932		QY								
23933										
23940		QY								
23941										
23942										
23943										
23950	QY	QY	YIZ PFA		UB WF		QY		UBW IZ	
23951										
23952										
23960										
23970										
23980										
23981			UBX							
23982										
23983					WF					
23984										
23990										

(19)

DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A01 Possum Kingdom Spillway TM 111-1	A02 Possum Kingdom Submerged Bucket TM 111-2	A03 Possum Kingdon Baffles TM 111-3	A04 St. Lucie Canal L&D TM 153-1	A05 Santee River Spillway TM 168-1	A06 Canton Spillway TM 190-1	A07 Bluestone Baffles TM 2-243	A08 Demopolis L&D TM 2-252	A09 Morganza Floodwy Contr Structure TM 2-326	A10 Jim Woodruff L&D TM 2-340	A11 Cheatham Emergency Dam TM 2-358	A12 Cheatham Spillway Gate TM 2-381	A13 New Cumberland L&D TM 2-386	A14 Gavin's Point Spillway TM 2-404	A15 Old River L Sill Multi-Leaf Gates TR 2-447 Rpt 1	A16 Old River L Sill Dnstr Ch Riprap TR 2-447 Rpt 2	A17 Old River L Sill Control Struct TR 2-447 Rpt 3	A18 Warrior Spillway TR 2-485	A19 Old River O'bank Panel Gates TR 2-491	A20 Old River Closure Dam TR 2-496
24000 STILLING BASIN (APRON)	U			UBW	CE					E						VR				
24100 Type		UB WE																		
24120																				
24140																				
24200 Apron				UVCB WYE	TUVCB WYIEZ	VE	QUVBW YIEZP		CBW	UVB WYI		UBW EF	UVB WYE	TUVB WYE	UBW	UBW	UC BW	UVW YI	U	
24210 Invert El	UVBW IE					QUVBW YIEZ		UVB WYI					E	VIE			VE	UVW YIE		
24220 Length	UVBW IE			VC YE		QUVBW YIEZ		UVB WYI	V	UVW YI		QUVY IEZF	UVW YE	VIE			VE			
24230 Width																				
24240 Slope	UVBW IE					QVY IEZ														
24250 Noted Items										QUVB WYI										
24300 End Sill	UVBW IE		E	UVCB WYE	TUVCB WYIEZ	QUVBW YIEZ	QUVBW YEZP		CBW				UVB WYE	V						
24310 Shape																				
24320 Height				E				UVB WYI				QUVY IEZF		UVB WIE		ER	VE			
24330 Noted Items							UVBWY IEZP	UB WY												
24400 Baffles			VBW IE	UVCB WYE	TUVCB WYIEZ	QUVBW YIEZP	QUVBW YIEZP		VCB WE				UVB WYE	VIE				UVW YI		
24410 Shape																	VE			
24411 Height																		UVW YIE		
24412 Width																		VE		
24413 Spacing			BWE															UVW YIE		
24420 Row(s)																	VE			
24421 Number																		UVW YIE	UV BW	
24422 Location														VIE				UVW YIE	UV BW	
24430 Noted Items							Z													
24500 Pier Extensions					TUVCB WYIEZ			VCN				QZF					UBW			
24510 Height																				
24520 Length																				
24530 Noted Items																				
24600 Training Walls	UVBW IE		VBW IE		VIE	VE			QVCB WYE					UBW			UCB WY	VE		
24610 Height																		C		
24620 Length																				
24630 Noted Items																				
24700 Riprap																				
24710 Bottom					UVCB WIE											ER				
24720 Side																				
24730 Size																				R
24740 Thickness																				
24750 Slope																				R
24760 Noted Items																				
(Continued)																				

20	A21	A22	A23	A24	A25	A26	A27	A28	A29	A30	A31	A32	A33	A34	A35	A36	A37	A38	A39	A40	A41	A42	A43	A44	A45
	W F George L&D TR 2-519	Jackson L&D TR 2-531	Dardanelle L&D TR 2-558	Markland Gates & Spillway TR 2-566	Greenup Gates & Spillway TR 2-572	Columbia Gates & Spillway TR 2-578	Maxwell/Opekiska Gates & Basins TR 2-579	New Cumberland Gates & Basin TR 2-585	Pike Island Stilling Basin TR 2-586	C&S Florida Proj Spillway TR 2-633	Millers Ferry Gates & Basin TR 2-643	Proctor Spillway TR 2-645	Arkansas R Dams Overflow Embank TR 2-650	Arkansas R Dams Lo-Head Spillway TR 2-655 & App A	Oahe Spillway TR 2-657	Belleville Stilling Basin TR 2-687	Barkley Spillway TR 2-689	Cannelton Spillway & Gates TR 2-710	Hannibal Spillway TR 2-731	Holt L&D TR 2-745	Hugo Spillway TR H-69-15	Copan Spillway TR H-70-09	Oakley Spillway TR H-70-13 & App	Arkansas R Dams Gate Vibration TR H-71-05	Miss R L&D 15 Spillway & Gates TR H-73-15
24000												QUB WY													
24100		UVW YI				UWY IE								UBW						U					
24120																									
24140																									
24200	UVWY IEZ	UVW YIE	UVB WYI	UVB WE	UV BW	UVB WYI	UVB WY	UVB WY	UV BW	UBW	UBW		UBW	UVBW YERZ	QUV WYI	UVB WYR	UVW YI	UVB WX	UV BW	UVCB WYI	UVCB WYI	UVB WYI	QUVC BWY		VI
24210	UVWY IEZ	UVW YIE	UVW YI	UV				UV		QY	UV			QUVB WRZ		UV BW		UR		UVW YI	UVB WYI				UBW
24220	UVWY IEZ							UV	V		UV			UVBW YERZ		UV	U	ER	UV			UVW YI	QUVC BWYR		UB WY
24230								UV																	
24240											UV														
24250														UBW											
24300			UVW YI	UVE	UV	ZP	UVB WY	UVB WY	V					UBW YER	QUV WYI	UVB WYR	U	UVB WE		UBW	UVW YI	UVW YI	UBW YR		UB WY
24310			UVB WYI																						
24320		U	UVB WYI	UE							UV			UV BW											
24330																									
24400				UVE	UV		UVB WY	UVB WY	V					UBW YER	QUV WYI	UVB WYR		UVB WE				UVW YI	UB WY		UBW
24410																									
24411				VE				U										UV				VI	QUVC BWYR		
24412																									
24413																									
24420																									
24421				VE																		VI			
24422				VE				U	UBW									UVR	UV				QUVC BWYR		
24430																							QUVC BWYR		
24500		U												UBW ER				X	UB WR						
24510																									
24520																		R							UB WY
24530																									
24600															UVW YI		C		UB WR	UBW					
24610	VY																				VCY				
24620	VY																					UVW YI			
24630																					VCY				
24700						VR								UBWR											
24710													QUVBW YERL			R		UB WR	UB WR		R				R
24720																									
24730													QVWY ERL	UER							R		R		R
24740													QVWY ERL								R				R
24750														R				UB WR					R		
24760																		R							R
(Continued)																									

(21) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A46 Aliceville Spillway TR H-74-10	A47 Columbus Spillway TR H-74-13	A48 Old River L Sill Proto Vibration TR H-76-15	A49 Old River L Sill Operation TR H-77-02	A50 Ozark Proto Spillway TR H-77-06	A51 Red River L&D 1 Spillway TR H-77-13	A52 Tenn-Tom Canal Spillways A&B TR H-78-21	A53 Barkley Proto Gate Vibr TR HL-79-08	A54 Grays Landing Spillway TR HL-81-13	A55 Barkley Gate & Bulkhead TR HL-83-12	A56 Baffle Piers Cavitation MP 2-154	A57 Navigation Dam Swellhead MP 2-468	A58 Spillway Gates Vibration MP 2-484	A59 Overflow Embank Riprap MP 2-552	A60 Vert Lift Gates Discharge MP 2-606	A61 Spwy Tow Curves Pressures MP 2-625	A62 Sbmrgd Rock Weir Propeller Wash MP 2-821	A63 Rend L Sub Dams Riprap Cover MP H-68-01	A64 Baffled Basins Design Trends MP H-69-01	A65 Baffle Piers Drag Forces MP H-70-04
24000 STILLING BASIN (APRON)																				
24100 Type									UVB WY							Z				
24120																				
24140																				
24200 Apron			UCB WY	UVW YZ		TUVCB WENR	UVCB WYI		UVBW YRX	UCB WS			X						QVX	UVBW YIFX
24210 Invert El	UB WY	UBW				UVC BW	UVC BWY			S										
24220 Length	UB WY					UVC BW			UB WY	S										U
24230 Width																			TQX	
24240 Slope	UB WY																			
24250 Noted Items	V															Z				Y
24300 End Sill						UVC BW			UVB WY	S									QVX	
24310 Shape																				
24320 Height																			VYX	U
24330 Noted Items																				
24400 Baffles						UVC BWN			UVB WY										QVX	UB WX
24410 Shape											EX									UVBWY IZPFX
24411 Height									UBW YRX										VYX	UVBWY IZFX
24412 Width																				UVWY IZFX
24413 Spacing																				UVWY IZFX
24420 Row(s)																				
24421 Number	UB WY																			UWY ZF
24422 Location	UBWY																		VYX	UVBWY IZPFX
24430 Noted Items											EX									
24500 Pier Extensions	UB WY	UBW				TUVCB WENR	UVCB WYIF			UB WS										
24510 Height										FA										
24520 Length										FA										
24530 Noted Items										F										
24600 Training Walls						UC BW														
24610 Height																				
24620 Length	CW																			
24630 Noted Items																				
24700 Riprap																		UB WI		
24710 Bottom														UBW YR						
24720 Side																				
24730 Size														R			R	R		
24740 Thickness																		R		
24750 Slope																				
24760 Noted Items																				
(Continued)																				

(22)	A66 Spillway Exit Ch Bottom Shape MP H-71-05	A67 Tainter Gates CE Project Data MP H-72-07	A68 Spillway Nappe Upper Surface MP H-73-04	A69 Spillway Crest Design Profile MP H-73-05	A70 Spillway Chute Flow Surface MP H-76-19	A71 Old River O'bank Outlet Channel MP HL-80-05	A72 Low-Ogee Crest Pressure Fluct CR H-71-01	A73 Ohio R L&D 37 Spillway Paper D	A74 RR Embankment O'flow Erosion Paper R	A75 St. Lucie Canal Spillways Paper 14	A76 St. Clair River Submerged Sills Paper 16	A77 Hastings Spillway STP 01	A78 Sand Dams O'flow Discharge STP 02	A79 Kiskiminitas 2 Spillway & Chnnl STP 03	A80 Beartrap Dam Chamber Silting STP 04	A81 Marmet L&D STP 05	A82 Miss R L&D 15 Dam & Spillway STP 07	A83 App A to STP 07 Model vs Proto STP 09	A84 Monongahela 4 Crest & Basin STP 12	A85 Roller Gates Discharge Coef STP 13	A86 Montgomery Is Channel & Spwy STP 14	A87 Winfield Channel & Spwy STP 15	A88 Miss R 5-5A-8 Roller Gt Coef STP 17	A89 Miss R L&D 26 Chnl & Cofferdam STP 20	A90 Peoria/La Grange Wicket Discharge STP 23
24000																									
24100	TUB WE	X								TUB WER															
24120																CN									
24140																CN									
24200	UBW	X					UBWY IZP		TUVB WYER			UVWY IER				UBW YIE	UB WY		TQUVB WYIEZ X	UC BW		UBW ER	QUV WYE	UVBW YIE	UBW
24210												UVW YI				UBW YE						TUB WE	QUY	TUVB WYE	
24220										TUB WER		UWY				UBW YE						E		UVE	
24230																									
24240																									
24250																			QVW YZ			TUB WE			
24300		X								TUB WER		VI				UBW YE			TQUVB WYIEZ X						
24310																									
24320																						E			
24330																									
24400		X										UVW YI				UBW YE			TQUVB WYIEZ X					UE	
24410																									
24411																									
24412																									
24413																									
24420																									
24421																									
24422																						TUB WE		UB WY	
24430																									
24500												U				UBW YE								UBW	
24510																									
24520																									
24530																									
24600		X																						CE	
24610																									
24620																									
24630																									
24700																									
24710	ER								TUVW YER													ER	QUY	UVE	
24720																									
24730									TUVW YER																
24740																									
24750	ER																								
24760																									
(Continued)																									

(23)

DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A91 Miss R L&D 20 Stilling Basin STP 24	A92 Miss R L&D 7 Spwy Culverts STP 29	A93 Chanoine Wickets Discharge Coef STP 30	A94 Miss R 5-5A-8 Tainter Gt Coef STP 31	A95 Miss R L&D 22 Stilling Basin STP 33	A96 Roller Gate Stilling Basins STP 36	A97 Sub Tainter Gate Coef & Basin STP 37	A98 App A to STP 13 Roller Gate Coef STP 39	A99 Miss R L&D 4 Gates & Basin STP 53	B01 Miss R L&D 1 Spillway Apron STP 63	B02 SAF Lower L&D Tntr Gt & Basin STP 69	B03 Roller Gates Pressures STP 77	B04 Bonneville Spillway Press BHL 3-1	B05 McNary Cffrdms & T-race BHL 20-1	B06 McNary Spillway & Gates BHL 21-1	B07 Ice Harbor Cffrdms & T-race BHL 22-1	B08 Ice Harbor Spillway BHL 31-1	B09 The Dalles Spillway & Basin BHL 55-1	B10 Bonneville Rev Still Basin BHL 65-1	B11 John Day Spwy & Dvrsn Gap BHL 97-1
24000 STILLING BASIN (APRON)																				
24100 Type																			UV YZ	
24120																				
24140																				
24200 Apron	QUBW YEX	UB WY	UBW	UBW	TUBW YEZL	TUVB WYIE	TUV BWE	UBW	TQUVB WYIER	UBW YER	UBW YEZ	Z		UVC NL	QUV WYI	VC YI	QUVBW YIZX	QUVB WYIZ	UVBW YIZX	UVBW YIEZ
24210 Invert El	UBW				TUVB WIE	TE	QYL		QUY									QUVB WYIZ		UB WY
24220 Length	UB WX				TUVB WIE	E	QYL		QY						UB WY			QUVB WYIZ		UB WY
24230 Width									QV YE											
24240 Slope																				
24250 Noted Items									TQUVB WYIE										UVBW YIZX	
24300 End Sill	UYX				VIE	TE			TQUVB WYIE	UW YE					Z		QUVB WYIZ		TQUVB WYIEZ	UVBW YIZ
24310 Shape					TUVB WIE	E											VIZ			Z
24320 Height	UBW				TUVB WIE											UC		QUVB WYIZ		
24330 Noted Items																				
24400 Baffles	QUBW YLX	QYE				TE	TQU YEL		TQUVB WYIE	TUBW YER	UBW YE			UZ	Z		QUVB WYIZ	UYZ	TQUVB WYIEZ X	UVBW YIZ
24410 Shape					TUVB WIE	E													UV YZ	Z
24411 Height						TU VE					UWY									
24412 Width																				
24413 Spacing																				
24420 Row(s)						TU VE												QUVB WYIZ	UV YZ	
24421 Number						E	TE		UER											
24422 Location					TUVB WIE	UB WE									UB WY					
24430 Noted Items															Z					
24500 Pier Extensions						TE			TQUB WYE											
24510 Height																				
24520 Length						UB WE										UC				Z
24530 Noted Items																				
24600 Training Walls									E						YL					
24610 Height																				
24620 Length																				
24630 Noted Items														UVC INL		UVC YI	UVB WIZ			
24700 Riprap						TUB WE														
24710 Bottom	UBX	E							UER	UBW YER										
24720 Side																				
24730 Size																				
24740 Thickness																				
24750 Slope																				
24760 Noted Items																				
(Continued)																				

(24)	B12 Lower Monumental Spillway & Basin BHL 104-1	B13 Little Goose Spillway & Basin BHL 114-1	B14 Bonneville Gate Lip Mod BHL 136-1	B15 Allegheny L&D 9 Spwy & Channel CIT A4L9	B16 Wicket Gate Operating Forces CIT C3	B17 Bluestone Baffle Pier Cav CIT B57R	B18 Bonneville Baffle Pier Cav CIT B6C	B19 Tuscaloosa Stilling Basin CIT T8	B20 Gallipolis Channel & Spwy CIT G35F	B21 Gallipolis Proto Currents HDG35R
24000										
24100				E				UVY IE		
24120										
24140										
24200	UBW YZ	TQUVB WYIEZ P		UVC BW	UBW	UBY IX	UBY IEX	UVY IE	UBW	
24210		TQUVB WYIEZ P		E				UVY IE		
24220				E				UVY IE		
24230										
24240										
24250										
24300	UVBW YIZ	UY		E				UVY IE		
24310										
24320										
24330		TQUVB WYIEZ P								
24400							UYI EX	UVY IE		
24410						UBY IX	UBI EX			
24411										
24412										
24413							U			
24420										
24421										
24422										
24430										
24500		UBW								
24510										
24520										
24530										
24600	UB WY									CN
24610										
24620										
24630										
24700										
24710										
24720										
24730										
24740										
24750										
24760										
(Continued)										

25 DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A01 Possum Kingdom Spillway TM 111-1	A02 Possum Kingdom Submerged Bucket TM 111-2	A03 Possum Kingdon Baffles TM 111-3	A04 St. Lucie Canal L&D TM 153-1	A05 Santee River Spillway TM 168-1	A06 Canton Spillway TM 190-1	A07 Bluestone Baffles TM 2-243	A08 Demopolis L&D TM 2-252	A09 Morganza Floodwy Contr Structure TM 2-326	A10 Jim Woodruff L&D TM 2-340	A11 Cheatham Emergency Dam TM 2-358	A12 Cheatham Spillway Gate TM 2-381	A13 New Cumberland L&D TM 2-386	A14 Gavin's Point Spillway TM 2-404	A15 Old River L Sill Multi-Leaf Gates TR 2-447 Rpt 1	A16 Old River L Sill Dnstr Ch Riprap TR 2-447 Rpt 2	A17 Old River L Sill Control Struct TR 2-447 Rpt 3	A18 Warrior Spillway TR 2-485	A19 Old River O'bank Panel Gates TR 2-491	A20 Old River Closure Dam TR 2-496
24000 STILLING BASIN (CONTINUED)																				
24900 Operation																				
24910 Pool El	UVBW IE		VBW IE	UVB WYE	TQUVC BWYIE Z	QVY IEZ	QUVBW YIEZP	UVB WYI		UVB WYI			UVW YE	UVB WI			UCB WY	UVW YI		
24920 TW El	UVBW IE		VBW IE	UVB WYE	TQUVC BWYIE Z	QUVBW YIEZ	QUVBW YIEZP	UVB WYI		UVB WYI		UB WE	UVB WYE	UVB WI			UVCB WYE	UVW YI	UV BW	
24930 Type Flow																				
24931 Free/Submerged					TQUVB WYIEZ			UVBW YIH	CBW	UVB WYI		UBW	UVB WYE					UVW YIE	U	
24932 Gated/Uncontrolled													UVB WYE					UVW YIE		
24933 Unit Discharge	UVBW IE		VBW IE	UVB WYE	TUVCB WYIEZ	QUVBW YIEZ	QUVBW YIEZP	UVB WYI		UVW YI			UVB WYE	UVB WI						R
24940 Gate Schedule	UBW																			
24941 Single					CE								E	UBW			UC			
24942 Multiple				UBW	TUVCB WYIEZ									UBW			UVC BWE	UVW YIE		
24943 Locations				UBW													UC			
24950 Gate Opening	U											UB WE	UVB WY	UBW				UWY		
24951 Uniform				UVB WYE																
24952 Variable					CE													UVW YIE		
24960 Gate Submergence				U								UB WE						VE		
24970 Gate Speed																				
24980 Other Factors																				
24981 Ice/Debris																				
24982 Loose Barges																				
24983 Waves																				
24984 Power Discharge																				
24990 Noted Items						VYI EZ														
25000 DOWNSTREAM CHANNEL																				
25100 Channel	UWB VI	UBW	BW	UVC BW	UCB WY	VCY IEZ		UVB WYI	VE	UVC WYI		QUVBW YIEL	UV WY	UVC BWY		UVB WR	UC BW	UW		UV ER
25110 Direction						VC														
25120 Shape								VC												
25121 Invert El	E	E	VIE	VE	TUVCB WYIE	VE			VE	E				TE		ER		UVW YIE		
25122 Width																				
25123 Side Slopes																				
25124 Bottom Slope																				
25130 Dikes																				
25140 Noted Items																				
25200 Guide/Guard Walls																				
25300 Riprap																				
25310 Bottom																ER				
25320 Side																ER				
25330 Size																VR		VR		
25340 Thickness																VR		VR		
25350 Slope																				
25360 Noted Items																VER				
25400 Noted Items														C						
(Continued)																				

(26)	A21 W F George L&D TR 2-519	A22 Jackson L&D TR 2-531	A23 Dardanelle L&D TR 2-558	A24 Markland Gates & Spillway TR 2-566	A25 Greenup Gates & Spillway TR 2-572	A26 Columbia Gates & Spillway TR 2-578	A27 Maxwell/Opekiska Gates & Basins TR 2-579	A28 New Cumberland Gates & Basin TR 2-585	A29 Pike Island Stilling Basin TR 2-586	A30 C&S Florida Proj Spillway TR 2-633	A31 Millers Ferry Gates & Basin TR 2-643	A32 Proctor Spillway TR 2-645	A33 Arkansas R Dams Overflow Embank TR 2-650	A34 Arkansas R Dams Lo-Head Spillway TR 2-655 & App A	A35 Oahe Spillway TR 2-657	A36 Belleville Stilling Basin TR 2-687	A37 Barkley Spillway TR 2-689	A38 Cannelton Spillway & Gates TR 2-710	A39 Hannibal Spillway TR 2-731	A40 Holt L&D TR 2-745	A41 Hugo Spillway TR H-69-15	A42 Copan Spillway TR H-70-09	A43 Oakley Spillway TR H-70-13 & App	A44 Arkansas R Dams Gate Vibration TR H-71-05	A45 Miss R L&D 15 Spillway & Gates TR H-73-15
24000																									
24900																									
24910	UVWY IEZ	UVW YI	UVB WYI			UVBW YIR		U		UBW	UV	QUB WY	QUVBW YERL	UVBW YERZ				U	R	UVW YI	UVCB WYI	UBW	QUVC BWYR		WY
24920	UVWY IEZ	UVW YIE	UVB WYI	UVB WE	UV BW	UVBWY IEZP	UVB WY	UVB WY	UV BW	UBW	UV BW	QUB WY	QUVBW YERL	UVBW YERZ	UVW	UVB WYR	UVW YI	UVB WRX	UVB WR	UVW YI	UVCB WYI	UBW	QUVC BWYR		
24930																									
24931		UVW YI	UVW YI	UBW	UBW	UVBWY IEZP	UBW	UVB WY	UV BW	UBW	UV BW		QUVBW YERL	UVBW ERZ	UVW	UVB WYR	UVW YI	UV	UV BW	UVW YI					
24932		UVW YI	UVW YI							UBW	UV			WYZ	V		UVW YI		UV	UVW YI					
24933	UVWY IEZ	UVW YI				UVW YI							QUVBW YERL	UV											
24940																		UV BW							
24941															UVW	UVB WYR			UV						UVB WIR
24942															UVW	UVB WYR									R
24943																									
24950			UBW	UVE	UV	UVBWY IEZP	UVB WY	UVB WY	V		UV			UVBW YERZ		UB WY	UVW YI	UVB WR					UVCB WYR		UVB WI
24951	VY	UVW YI													UVW				UV						
24952															UVW										
24960				UVE				U															UB WY		
24970																									
24980																									
24981																UVB WYR		UB WX							
24982																									
24983																									
24984	VY																			VCN					
24990																									
25000																									
25100	UVCWY IENZH	UVW YI	UVCB WYI			UVBW YIE			V	UBW	UV BW			UVB WYR	VW YI		TUVC WYI			UVCB WYIN	UVW YI	TUVC BWIR	UVCB WYR		VI
25110																									
25120			VC			UVBWY IR									QY					VCN					
25121	UVWY IEZ	UE	UVW YI	UVE						QY			WY ER	UBW ER				E		UVW YI					
25122																									
25123															UVW YI										
25124																									
25130																				VCN					
25140															QUV WYI							TUVC BWYR			
25200																	TVC WY			CN					
25300															UVW		VC								
25310																									VER
25320																						TR			
25330																							R		ER
25340																									ER
25350																									ER
25360																									R
25400																					CBW		UVC BWR		
(Continued)																									

(27) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A46 Aliceville Spillway TR H-74-10	A47 Columbus Spillway TR H-74-13	A48 Old River L Sill Proto Vibration TR H-76-15	A49 Old River L Sill Operation TR H-77-02	A50 Ozark Proto Spillway TR H-77-06	A51 Red River L&D 1 Spillway TR H-77-13	A52 Tenn-Tom Canal Spillways A&B TR H-78-21	A53 Barkley Proto Gate Vibr TR HL-79-08	A54 Grays Landing Spillway TR HL-81-13	A55 Barkley Gate & Bulkhead TR HL-83-12	A56 Baffle Piers Cavitation MP 2-154	A57 Navigation Dam Swellhead MP 2-468	A58 Spillway Gates Vibration MP 2-484	A59 Overflow Embank Riprap MP 2-552	A60 Vert Lift Gates Discharge MP 2-606	A61 Spwy Tow Curves Pressures MP 2-625	A62 Sbmrgd Rock Weir Propeller Wash MP 2-821	A63 Rend L Sub Dams Riprap Cover MP H-68-01	A64 Baffled Basins Design Trends MP H-69-01	A65 Baffle Piers Drag Forces MP H-70-04
24000 STILLING BASIN (CONTINUED)																				
24900 Operation																				
24910 Pool El				UV WZ			UBW		UVB WYX					UB WR		Z		UB WI	QVX	
24920 TW El	UBW	UBW		UVW YZ		UVC BW	UVB WYI		UVB WYX	UB WS	X			UB WR				UB WI	QV YX	UVBWY IZFX
24930 Type Flow																				
24931 Free/Submerged				UVW YZ		UVC BW			UVB WYX	UB WS				UB WR						
24932 Gated/Uncontrolled	UB WY	UBW		UW YZ		UVC BW	UVCB WYI													
24933 Unit Discharge											X			UB WR		Z			TQ VX	VY IX
24940 Gate Schedule																				
24941 Single	UB WY	UBW				TUVC BWER	UVC BWI													
24942 Multiple	UBW					UVC BW														
24943 Locations		UBW				UC BW														
24950 Gate Opening	UB WY			UV WZ		UVC BWR	UVCB WYI			UB WS			X							
24951 Uniform																				
24952 Variable																				
24960 Gate Submergence																				
24970 Gate Speed																				
24980 Other Factors																				
24981 Ice/Debris									UBW YX											
24982 Loose Barges						N														
24983 Waves																				
24984 Power Discharge																				
24990 Noted Items											X								VYX	UVBWY IZPFX
25000 DOWNSTREAM CHANNEL																				
25100 Channel	VC YN			UV CW		V	UVCB WYI		UVB WYX											
25110 Direction																				
25120 Shape	C											QYL								
25121 Invert El	C																			
25122 Width																				
25123 Side Slopes																				
25124 Bottom Slope																				
25130 Dikes	VCE NR																			
25140 Noted Items																				
25200 Guide/Guard Walls																				
25300 Riprap	ER	R																		
25310 Bottom		VR				TV ER			R											
25320 Side		VR																		
25330 Size		VR				TV ER			R											
25340 Thickness		VR				TV ER														
25350 Slope																				
25360 Noted Items																				
25400 Noted Items																				
(Continued)																				

(28)	A66 Spillway Exit Ch Bottom Shape MP H-71-05	A67 Tainter Gates CE Project Data MP H-72-07	A68 Spillway Nappe Upper Surface MP H-73-04	A69 Spillway Crest Design Profile MP H-73-05	A70 Spillway Chute Flow Surface MP H-76-19	A71 Old River O'bank Outlet Channel MP HL-80-05	A72 Low-Ogee Crest Pressure Fluct CR H-71-01	A73 Ohio R L&D 37 Spillway Paper D	A74 RR Embankment O'flow Erosion Paper R	A75 St. Lucie Canal Spillways Paper 14	A76 St. Clair River Submerged Sills Paper 16	A77 Hastings Spillway STP 01	A78 Sand Dams O'flow Discharge STP 02	A79 Kiskiminitas 2 Spillway & Chnnl STP 03	A80 Beartrap Dam Chamber Silting STP 04	A81 Marmet L&D STP 05	A82 Miss R L&D 15 Dam & Spillway STP 07	A83 App A to STP 07 Model vs Proto STP 09	A84 Monongahela 4 Crest & Basin STP 12	A85 Roller Gates Discharge Coef STP 13	A86 Montgomery Is Channel & Spwy STP 14	A87 Winfield Channel & Spwy STP 15	A88 Miss R 5-5A-8 Roller Gt Coef STP 17	A89 Miss R L&D 26 Chnl & Cofferdam STP 20	A90 Peoria/La Grange Wicket Discharge STP 23
24000																									
24900																									
24910							UBWY IZP		TUVB WYR	TUB WER		UWY							TQUVB WYIEZ X				UV WE	UVB WYE	
24920							UBW YIP		TUVB WYR	TUB WER		UWY				UBW YIE			TQUVB WYIEZ X				UV WE	UVB WYE	
24930																									
24931																			TQUVB WYIEZ X						
24932																									
24933							UBWY IZP			TUB WER						UBW YE			TQUVB WYIEZ						
24940																									
24941																UBW YE									
24942												ER				UBW YE								UBW YE	UBW
24943																									UBW
24950												ER				UBW YE	UB WY					UBW			
24951																									
24952																									
24960																							UV WE		
24970																									
24980																									
24981																									
24982																									
24983																									
24984																									
24990							P																		
25000																									
25100	TUB WE											UVCW YIER		QCY EL		QUCBW YIENL	QUCB WYNL	QV YL			QVCY IENL	QVCY INL	UV WE	UVCY IEN	
25110																									
25120	UBW ER															QCY INL									
25121														QCY EL		UBW YE			TE		QCY ENL	TUB WER	QUV WYE	TC ER	E
25122																QYL									
25123																									
25124																							QUY		
25130																								TC YE	
25140																QVC YNL	CN		TE		QC YL				
25200														C								VC IN			
25300	UBW																							CER	
25310						UBW YER				TUB WER													QUY		
25320																									
25330																									
25340																									
25350																									
25360						UBW YER																			
25400																QVY NL								TC ER	
(Continued)																									

(29) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A91 Miss R L&D 20 Stilling Basin STP 24	A92 Miss R L&D 7 Spwy Culverts STP 29	A93 Chanoine Wickets Discharge Coef STP 30	A94 Miss R 5-5A-8 Tainter Gt Coef STP 31	A95 Miss R L&D 22 Stilling Basin STP 33	A96 Roller Gate Stilling Basins STP 36	A97 Sub Tainter Gate Coef & Basin STP 37	A98 App A to STP 13 Roller Gate Coef STP 39	A99 Miss R L&D 4 Gates & Basin STP 53	B01 Miss R L&D 1 Spillway Apron STP 63	B02 SAF Lower L&D Tntr Gt & Basin STP 69	B03 Roller Gates Pressures STP 77	B04 Bonneville Spillway Press BHL 3-1	B05 McNary Cffrdms & T-race BHL 20-1	B06 McNary Spillway & Gates BHL 21-1	B07 Ice Harbor Cffrdms & T-race BHL 22-1	B08 Ice Harbor Spillway BHL 31-1	B09 The Dalles Spillway & Basin BHL 55-1	B10 Bonneville Rev Still Basin BHL 65-1	B11 John Day Spwy & Dvrsn Gap BHL 97-1
24000 STILLING BASIN (CONTINUED)																				
24900 Operation																				
24910 Pool El	QUB YEX			UBW	UVYI EZL		UBW		TQUB WYE	UBW YER	UWY				Z	VC YI	UB WX	QUVB WYIZ	UVBW YIZ	UVBW YIEZ
24920 TW El	QUB YEX			UBW	UVBWY IEZL	TUV YE	TUB WE	UBW	TQUB WYER	UBW YER	UB WY	Z		UZ	QUVBW YIZL	VC YI	QUVBW YIZX	QUVB WYIZ	UVBW YIZ	UVBW YIEZ
24930 Type Flow																				
24931 Free/Submerged		UB WY			UVBWY IEZL		UBW	UBW	TQUB WYE											U
24932 Gated/Uncontrolled															QU ZY			QUVB WYIZ		
24933 Unit Discharge									QUY											
24940 Gate Schedule														UVC NZL	YZ PL	VC YI	QUVB WYIZ	QUVB WYIZ	Z	Z
24941 Single																				CW
24942 Multiple																				
24943 Locations																				
24950 Gate Opening	QUB YEX			UBW	UVB WIE	TUV YE	TUB WE		TQUB WYER		UB WY	Z		UCN ZL	UBWY ZPL	VC YI	QUVBW YIZX	QUVB WYIZ	UVBW YIZ	Z
24951 Uniform																				
24952 Variable																				
24960 Gate Submergence					VIE	TU VE	TUB WE		TQUB WYE		UB WY	Z		UZ						
24970 Gate Speed																				
24980 Other Factors																				
24981 Ice/Debris						TU VE											UX			UBW YIE
24982 Loose Barges																				
24983 Waves																				
24984 Power Discharge														UVC NL		VC YI				V
24990 Noted Items										UBW					UVWY IZP					
25000 DOWNSTREAM CHANNEL																				
25100 Channel	UY EX				UVB WIE				TQUVC BWYIE L	QUBW YER				VC YN		TUVC WYSI	UB WY		UVB WYI	UVB YIE
25110 Direction																				
25120 Shape														VC YN		VY				
25121 Invert El	E				TUVB WIE	TU VE	VE		TQUVB WYIER	TUBW YER	E				QUVW YIZ	VCY IE	QUVB WYIZ	QUVB WYIZ	TE	UVBW YIZ
25122 Width																				
25123 Side Slopes																				
25124 Bottom Slope																				
25130 Dikes																C				
25140 Noted Items														UVC YIN		VC YI				
25200 Guide/Guard Walls														VCN						
25300 Riprap																				
25310 Bottom																				
25320 Side																				
25330 Size																				
25340 Thickness																				
25350 Slope																				
25360 Noted Items																				
25400 Noted Items														UV CI		TUCW YSI				
(Continued)																				

30	B12 Lower Monumental Spillway & Basin BHL 104-1	B13 Little Goose Spillway & Basin BHL 114-1	B14 Bonneville Gate Lip Mod BHL 136-1	B15 Allegheny L&D 9 Spwy & Channel CIT A4L9	B16 Wicket Gate Operating Forces CIT C3	B17 Bluestone Baffle Pier Cav CIT B57R	B18 Bonneville Baffle Pier Cav CIT B6C	B19 Tuscaloosa Stilling Basin CIT T8	B20 Gallipolis Channel & Spwy CIT G35F	B21 Gallipolis Proto Currents HDG35R
24000										
24900										
24910	UVBW YIZ	QUVBW YIEZP		UVC BW	UBW	UBY IX	UBY IX	UYI		
24920	UVBW YIZ	QUVBW YIEZP		UVC BW	UBW	UBY IX	UBY IX	UYI		
24930										
24931										
24932										
24933										
24940										
24941										
24942										
24943										
24950	UVBW YIZ	QUVBW YIEZP			UBW		UBY IX			
24951										
24952										
24960										
24970										
24980										
24981							VIE			
24982										
24983										
24984	UB WY									
24990						UB IX	UBI EX			
25000										
25100	UVB WYI	UBW YE		DV CN				UVY IE		VC NX
25110										
25120	UVBW YIZ	TQUVB WYIEZ P								
25121		UBW YE		E				UVY IE		
25122										
25123										
25124										
25130										
25140										
25200										
25300										
25310										
25320										
25330										
25340										
25350										
25360										
25400										
(Continued)										

(31) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A01 Possum Kingdom Spillway TM 111-1	A02 Possum Kingdom Submerged Bucket TM 111-2	A03 Possum Kingdon Baffles TM 111-3	A04 St. Lucie Canal L&D TM 153-1	A05 Santee River Spillway TM 168-1	A06 Canton Spillway TM 190-1	A07 Bluestone Baffles TM 2-243	A08 Demopolis L&D TM 2-252	A09 Morganza Floodwy Contr Structure TM 2-326	A10 Jim Woodruff L&D TM 2-340	A11 Cheatham Emergency Dam TM 2-358	A12 Cheatham Spillway Gate TM 2-381	A13 New Cumberland L&D TM 2-386	A14 Gavin's Point Spillway TM 2-404	A15 Old River L Sill Multi-Leaf Gates TR 2-447 Rpt 1	A16 Old River L Sill Dnstr Ch Riprap TR 2-447 Rpt 2	A17 Old River L Sill Control Struct TR 2-447 Rpt 3	A18 Warrior Spillway TR 2-485	A19 Old River O'bank Panel Gates TR 2-491	A20 Old River Closure Dam TR 2-496
25000 DOWNSTREAM CHANNEL (CONT'D)																				
25900 Operation																				
25910 Pool El	VIE	E	VIE	VCE	TUVBW YIEZ	VE			VE	UVW YIE				TCE		ER	UC BW	UW		
25920 TW El	VIE	E	VIE	VCE	TUVBW YIEZ	VC YE		VC	VE	UVC WYI		QYL		TCE		VER	UC BW	UW		
25930 Type Flow										VC										
25931 Free/Submerged					TUVBW YIEZ				V	UVW YI										
25932 Gated/Uncontrolled																				
25933 Unit Discharge	VIE	E	VIE	VCE	TUVBW YIEZ	VC YE			VE	UVW YI				TE		VER		VR		
25940 Gate Schedule										VCE										
25941 Single					CE												UC			
25942 Multiple					TUVCB WYIEZ												UC BW			
25943 Location																	UC			
25950 Gate Opening										VCE										
25951 Uniform				VCE																
25952 Variable					TCE															
25960 Gate Submergence																				
25970 Gate Speed																				
25980 Other Factors																				
25981 Ice/Debris																				
25982 Loose Barges																				
25983 Waves																				
25984 Power Discharge														C						
25990 Noted Items						VE														
NOTED ITEMS																				
21140 Upstream approach channel																				
21460 Upstream approach riprap																				
21500 Upstream approach misc.																				
21990 Upstream approach operation																				
22240 Control sill shape		Submerged bucket				Sluice outlet				Dike section										Rock dam
22800 Control sill misc.				Lock chamber		Pier extension														Seal cover
22990 Control sill operation				Flow thru lock		Sluice flow														Constr proce-dure
23700 Gates & bulkheads misc.				Orifice gates								Seals & vents			Air vents					
23990 Gates & bulkhead operation																				
24250 Still basin apron										Flip bucket										
24330 Still basin end sill							Scndry weir	Notched sill												
24430 Still basin baffles							Location in row													
24530 Still basin pier ext																				
24630 Still basin tr walls																				
24760 Still basin riprap																				
24990 Still basin operation						Sluice flow														
25140 Downstream channel geometry																				
25360 Downstream channel riprap																Filter blanket				
25400 Downstream channel misc.														Pwrhs tailrace						
25990 Downstream channel operation						Sluice flow														
X OTHER TYPES OF DATA										Gate operation schedule		Air demand								Seep-age flow

32	A21 W F George L&D TR 2-519	A22 Jackson L&D TR 2-531	A23 Dardanelle L&D TR 2-558	A24 Markland Gates & Spillway TR 2-566	A25 Greenup Gates & Spillway TR 2-572	A26 Columbia Gates & Spillway TR 2-578	A27 Maxwell/Opekiska Gates & Basins TR 2-579	A28 New Cumberland Gates & Basin TR 2-585	A29 Pike Island Stilling Basin TR 2-586	A30 C&S Florida Proj Spillway TR 2-633	A31 Millers Ferry Gates & Basin TR 2-643	A32 Proctor Spillway TR 2-645	A33 Arkansas R Dams Overflow Embank TR 2-650	A34 Arkansas R Dams Lo-Head Spillway TR 2-655 & App A	A35 Oahe Spillway TR 2-657	A36 Belleville Stilling Basin TR 2-687	A37 Barkley Spillway TR 2-689	A38 Cannelton Spillway & Gates TR 2-710	A39 Hannibal Spillway TR 2-731	A40 Holt L&D TR 2-745	A41 Hugo Spillway TR H-69-15	A42 Copan Spillway TR H-70-09	A43 Oakley Spillway TR H-70-13 & App	A44 Arkansas R Dams Gate Vibration TR H-71-05	A45 Miss R L&D 15 Spillway & Gates TR H-73-15
25000																									
25900																									
25910	UVCWY IENZ	UVW YI	UVW YI			VR					UV		WY ER	UVB WER						UVW YI		TUVC BWYR	UVC BWR		
25920	UVCWY IENZ	UVW YIE	UVW YI			UVW YIE			V		UV BW		WY ER	UVB WER	QY		TUVC WYI			UVCW YIN		TUVC BWYR	UCB WR		
25930																									
25931		UVW YI	UVW YI			UVW YIE			V		UV BW		WY ER	UVB WER			UVW YI			UVW YI					
25932		UVW YI	UVW YI								UV						UVW YI			UVW YI					
25933	UVWY IEZ	UVW YI											WY ER												
25940																									
25941																									VI ER
25942	VCN		VC												UVW										VER
25943	VCN		VC																						
25950						UVW YI			V	UBW	UV			UVB WER			TUVC WYIN			N			UVC BWR		VI
25951	VCN	UVW YI													UVW										
25952															UVW										
25960																									
25970																									
25980																									
25981																									VR
25982																									
25983																									
25984	VC NH		VC														TVC			VC					
25990			VC														TVC			C					
21140																	Railroad embankment				Abutment shape		Abutment shape		
21460																									
21500			Guide booms																						
21990																									
22240	Divrsn notch	Air vents		Cheatham sill									Riprap cover							Diversion weir					
22800			Project layout																						
22990																									
23700				Gate seals	Air vents													Trunnion elevation							Gate struct
23990					Air vents																				
24250																									
24330																									
24430																							Chute blocks		
24530																									
24630																					Berm elev				
24760																		Blanket length							Cover blocks
24990																									
25140															Channel roughness							Levee			
25360																									Concr blocks
25400																					Bridges		Bridge		
25990			Lock discharge														Lock discharge			Trib inflow					
X			Debris accumulation	Seal pressure		Debris accumulation												Debris removal							Ice/debris removal

(33) DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A46 Allceville Spillway TR H-74-10	A47 Columbus Spillway TR H-74-13	A48 Old River L Sill Proto Vibration TR H-76-15	A49 Old River L Sill Operation TR H-77-02	A50 Ozark Proto Spillway TR H-77-06	A51 Red River L&D 1 Spillway TR H-77-13	A52 Tenn-Tom Canal Spillways A&B TR H-78-21	A53 Barkley Proto Gate Vibr TR HL-79-08	A54 Grays Landing Spillway TR HL-81-13	A55 Barkley Gate & Bulkhead TR HL-83-12	A56 Baffle Piers Cavitation MP 2-154	A57 Navigation Dam Swellhead MP 2-468	A58 Spillway Gates Vibration MP 2-484	A59 Overflow Embank Riprap MP 2-552	A60 Vert Lift Gates Discharge MP 2-606	A61 Spwy Tow Curves Pressures MP 2-625	A62 Sbmrgd Rock Weir Propeller Wash MP 2-821	A63 Rend L Sub Dams Riprap Cover MP H-68-01	A64 Baffled Basins Design Trends MP H-69-01	A65 Baffle Piers Drag Forces MP H-70-04
25000 DOWNSTREAM CHANNEL (CONT'D)																				
25900 Operation																				
25910 Pool El				UVW			UC BW		UVBW YRX											
25920 TW El	VCN			UVW		V	UVCB WYI		UVBW YRX											
25930 Type Flow																				
25931 Free/Submerged				UVW					UVBW YRX											
25932 Gated/Uncontrolled	Y			UW			UVCB WYI													
25933 Unit Discharge																				
25940 Gate Schedule																				
25941 Single	VC YN	VR		UW		V	UVB WI													
25942 Multiple																				
25943 Location						TER														
25950 Gate Opening	VC YN	VR		UVW		TV ER	UVCB WYI													
25951 Uniform																				
25952 Variable																				
25960 Gate Submergence																				
25970 Gate Speed																				
25980 Other Factors																				
25981 Ice/Debris									UBW YX											
25982 Loose Barges																				
25983 Waves																				
25984 Power Discharge																				
25990 Noted Items																				
NOTED ITEMS																				
21140 Upstream approach channel						Abutment radius														
21460 Upstream approach riprap																				
21500 Upstream approach misc.																				
21990 Upstream approach operation																				
22240 Control sill shape														Riprap cover		Flip bkt	Rock weir	Riprap cover		
22800 Control sill misc.			Bridge									Weir & o'bank flow					Cap blocks			
22990 Control sill operation												Flow thru lock					Propeller dir & dist			
23700 Gates & bulkheads misc.			Dog mech	Gate as weir						Bottom girder			Single leaf							
23990 Gates & bulkhead operation										Units on crest										
24250 Still basin apron	Fixed crest apron															Toe curve				Apron rough-ness
24330 Still basin end sill																				
24430 Still basin baffles											Prototype baffle									
24530 Still basin pier ext										Pier baffles										
24630 Still basin tr walls																				
24760 Still basin riprap																				
24990 Still basin operation											Velocity & pressure								Froude No.	Froude No.
25140 Downstream channel geometry																				
25360 Downstream channel riprap																				
25400 Downstream channel misc.																				
25990 Downstream channel operation																				
X OTHER TYPES OF DATA			Temper-ature						Ice pass-age		Cavi-tation		Debris on gate						Design data	Hydr jump loc & length

34	A66 Spillway Exit Ch Bottom Shape MP H-71-05	A67 Tainter Gates CE Project Data MP H-72-07	A68 Spillway Nappe Upper Surface MP H-73-04	A69 Spillway Crest Design Profile MP H-73-05	A70 Spillway Chute Flow Surface MP H-76-19	A71 Old River O'bank Outlet Channel MP HL-80-05	A72 Low-Ogee Crest Pressure Fluct CR H-71-01	A73 Ohio R L&D 37 Spillway Paper D	A74 RR Embankment O'flow Erosion Paper R	A75 St. Lucie Canal Spillways Paper 14	A76 St. Clair River Submerged Sills Paper 16	A77 Hastings Spillway STP 01	A78 Sand Dams O'flow Discharge STP 02	A79 Kiskiminitas 2 Spillway & Chnnl STP 03	A80 Beartrap Dam Chamber Silting STP 04	A81 Marmet L&D STP 05	A82 Miss R L&D 15 Dam & Spillway STP 07	A83 App A to STP 07 Model vs Proto STP 09	A84 Monongahela 4 Crest & Basin STP 12	A85 Roller Gates Discharge Coef STP 13	A86 Montgomery Is Channel & Spwy STP 14	A87 Winfield Channel & Spwy STP 15	A88 Miss R 5-5A-8 Roller Gt Coef STP 17	A89 Miss R L&D 26 Chnl & Cofferdam STP 20	A90 Peoria/La Grange Wicket Discharge STP 23
25000																									
25900																									
25910						UB WY				TUB WER				C		QVC YNL		QV YL	TE		QCY NL	QVCY INL		TUVC YIER	
25920										TUB WER		ER		QCY EL		QUVCB WYIEN L		QV YL	TE		QCY NL	QVCY INL		TUVC YIER	
25930																					QYL				
25931																QYL		VY			QYL				
25932																									
25933						UBW YR				TUB WER						UBW YE			TE		QYL				
25940														CE		CN		YL			QCY NL				
25941																QUBW YEL									
25942												ER				QUBW YEL	CN				QYL	VC IN		TVE	
25943												C					CN				QYL	VC IN		TE	
25950												ER		CE		UCBW YEN	UB WY	YL			QCY NL	TQUVC BWYIE NL			
25951																									
25952																									
25960																									
25970																									
25980																									
25981																									
25982																									
25983																									
25984																CIN						VC IN			
25990	E															CN		VYL				QYL			
																	Island								
21140																									
21460																									
21500																	Lock location								
21990																	Lock disch	Lock disch				Lock gates			
22240		Design head			Side walls	Surface protection					Rows of piling		Curb on sill						Proto-type						
22800	Outlet conduit													Lock site	Cul-verts	Chanoine weir					Fixed & skim weirs				Abut-ment con-duits
22990							Boundary layer trip								Culv valve	Flow thru lock						Model scale		Lock gates	
23700															Leaf chamber							Flap gate			B'fly valve
23990															System operation										
24250																			Loose slabs			Poirre dam sill			
24330																									
24430																									
24530																									
24630																									
24760																									
24990							Boundary layer trip																		
25140																Sgl lock str	Lock location		Proto-type		Training walls				
25360						Gabi-ons																			
25400																Lock wall extension								Bridge piers	
25990	Prototype operation															Flow thru lock		Lock disch				Lock gates			
X		Design data											Seepage				Surface debris		Internal press &/or velocity						

(35)

DESIGN AND OPERATIONAL VARIABLES / PROJECT AND REPORT	A91 Miss R L&D 20 Stilling Basin STP 24	A92 Miss R L&D 7 Spwy Culverts STP 29	A93 Chanoine Wickets Discharge Coef STP 30	A94 Miss R 5-5A-8 Tainter Gt Coef STP 31	A95 Miss R L&D 22 Stilling Basin STP 33	A96 Roller Gate Stilling Basins STP 36	A97 Sub Tainter Gate Coef & Basin STP 37	A98 App A to STP 13 Roller Gate Coef STP 39	A99 Miss R L&D 4 Gates & Basin STP 53	B01 Miss R L&D 1 Spillway Apron STP 63	B02 SAF Lower L&D Tntr Gt & Basin STP 69	B03 Roller Gates Pressures STP 77	B04 Bonneville Spillway Press BHL 3-1	B05 McNary Cffrdms & T-race BHL 20-1	B06 McNary Spillway & Gates BHL 21-1	B07 Ice Harbor Cffrdms & T-race BHL 22-1	B08 Ice Harbor Spillway BHL 31-1	B09 The Dalles Spillway & Basin BHL 55-1	B10 Bonneville Rev Still Basin BHL 65-1	B11 John Day Spwy & Dvrsn Gap BHL 97-1
25000 DOWNSTREAM CHANNEL (CONT'D)																				
25900 Operation																				
25910 Pool El					TUVB WIE				TQUVC BWYIE	TUBW YER				VCY		UVC YI		Z	TUVB WYIE	UVBW YIE
25920 TW El					TUVB WIE	TU VE	TU VE		TQUVC BWYIE L	TQUB WYER	E			UVC YNL		TUVC WYSI	UB WY	Z	TUVB WYIE	UVBW YIE
25930 Type Flow																C				
25931 Free/Submerged					TUVB WIE	TE	TVE		TQUB WYE											
25932 Gated/Uncontrolled																		Z		
25933 Unit Discharge																				
25940 Gate Schedule									QVCY IEL					UVC YNL		UVC YI	UB WY	Z		
25941 Single						TUB WE														
25942 Multiple						TE														
25943 Location																				
25950 Gate Opening					TUVB WIE	TU VE	TU VE		TQUVC BWYIE L		E			UVC YNL		TUVC WYSI	UB WY	Z	TUVB WYIE	
25951 Uniform																				
25952 Variable						E														
25960 Gate Submergence					TUVB WIE	TU VE	TU VE		TQUB WYE		E									
25970 Gate Speed																				
25980 Other Factors																				
25981 Ice/Debris						TUV BWE														UE
25982 Loose Barges																				
25983 Waves																				
25984 Power Discharge														VC YN		TUVC WYSI				
25990 Noted Items										UBW				UV CI		TUCW YSI				
NOTED ITEMS																				
21140 Upstream approach channel																				
21460 Upstream approach riprap																				
21500 Upstream approach misc.																				
21990 Upstream approach operation																				
22240 Control sill shape																				Dvrsn gap
22800 Control sill misc.		Cul-verts				Gate recess			Dike spwy						Proto-type					Stop-log slots
22990 Control sill operation													Proto-type		Gate leaf					
23700 Gates & bulkheads misc.			Wicket gap				End & trash shields					Seal & clearance			Gate slots					
23990 Gates & bulkhead operation					Water level inside gate		Water level inside gate						Proto-type		Gate leaf					
24250 Still basin apron									Profile shape										Bucket	
24330 Still basin end sill																				
24430 Still basin baffles															Proto-type					
24530 Still basin pier ext																				
24630 Still basin tr walls														Fish ladder		Fish ladder	Fish way			
24760 Still basin riprap																				
24990 Still basin operation										Proto-type					Gate leaf					
25140 Downstream channel geometry														Tail-race		Tail-race				
25360 Downstream channel riprap																				
25400 Downstream channel misc.														Pump intake		Lock outlet				
25990 Downstream channel operation														Pump flow		Lock operation				
X OTHER TYPES OF DATA	Hydr jump location & depth		Leakage					Leakage			Air de-mand	Water level inside gate	Prototype crest profile				Debris in basin	Debris pas-sage	Prototype cavitation	

36	B12 Lower Monumental Spillway & Basin BHL 104-1	B13 Little Goose Spillway & Basin BHL 114-1	B14 Bonneville Gate Lip Mod BHL 136-1	B15 Allegheny L&D 9 Spwy & Channel CIT A4L9	B16 Wicket Gate Operating Forces CIT C3	B17 Bluestone Baffle Pier Cav CIT B57R	B18 Bonneville Baffle Pier Cav CIT B6C	B19 Tuscaloosa Stilling Basin CIT T8	B20 Gallipolis Channel & Spwy CIT G35F	B21 Gallipolis Proto Currents HDG35R
25000										
25900										
25910		UVBW YIE		VC EN				UVY IE		VC NX
25920		QUVB WYIE		VC EN				UVY IE		VC NX
25930										
25931										
25932										
25933										
25940										VC NX
25941										
25942										
25943										
25950		UVBW YIE								VC NX
25951										
25952										
25960										
25970										
25980										
25981										
25982										
25983										
25984		UBW								VC NX
25990										
21140									Cells	
21460										
21500		Abut-ment								
21990										
22240										
22800	Abut-ments		Bridge clear-ance	Abutment wall			Gate slots			
22990										
23700					Tripping action					
23990										
24250										
24330		Bucket teeth								
24430										
24530										
24630										
24760										
24990						Vacuum test	Vacuum test			
25140										
25360										
25400										
25990										
X			Debris			Cavi-tation	Cavi-tation			Proto data

BIBLIOGRAPHY TO APPENDIX C

A01. US Army Engineer Waterways Experiment Station. 1936. "Model Studies of the Spillway and Stilling Basin for the Possum Kingdom Dam and Plates," Technical Memoranda 111-1, Vicksburg, Miss.

A02. US Army Engineer Waterways Experiment Station. 1936. "Additional Model Studies of the Stilling Basin for the Possum Kingdom Dam and Plates," Technical Memoranda 111-2, Vicksburg, Miss.

A03. US Army Engineer Waterways Experiment Station. 1937. "Model Study of Baffles for the Spillway Apron of the Possum Kingdom Dam and Plates," Technical Memoranda 111-3, Vicksburg, Miss.

A04. US Army Engineer Waterways Experiment Station. 1939. "Model Study of the Spillway for New Lock and Dam No. 1, St. Lucie Canal, Florida," Technical Memoranda 153-1, Vicksburg, Miss.

A05. US Army Engineer Waterways Experiment Station. 1940. "Model Study of the Spillway for the Santee River Dam," Technical Memoranda 168-1, Vicksburg, Miss.

A06. US Army Engineer Waterways Experiment Station. 1942. "Model Studies of the Spillway and Integral Sluices for the Canton Dam, North Canadian River, Oklahoma," Technical Memoranda 290-2, Vicksburg, Miss.

A07. US Army Engineer Waterways Experiment Station. 1948. "A Laboratory Development of Cavitation-Free Baffle Piers, Bluestone Dam, New River, West Virginia," Technical Memoranda 2-243, Vicksburg, Miss.

A08. US Army Engineer Waterways Experiment Station. 1948. "Spillway and Lock Approach Currents, Demopolis Lock and Dam, Tombigbee River, Alabama; Model Investigation," Technical Memoranda 2-252, Vicksburg, Miss.

A09. US Army Engineer Waterways Experiment Station. 1951. "Morganza Floodway Control Structure, Mississippi River; Hydraulic Model Investigation," Technical Memoranda 2-326, Vicksburg, Miss.

A10. US Army Engineer Waterways Experiment Station. 1952. "Spillway and Lock Approach, Jim Woodruff Dam, Apalachicola River, Florida; Model Investigation," Technical Memoranda 2-340, Vicksburg Miss.

A11. US Army Engineer Waterways Experiment Station. 1953. "Upstream Emergency Dam, Cheatham Lock, Cumberland River, Tennessee; Hydraulic Model Investigation," Technical Memoranda 2-358, Vicksburg, Miss.

A12. US Army Engineer Waterways Experiment Station. 1954. "Submergible-type Tainter Gate for Spillway, Cheatham Lock and Dam, Cumberland River, Tennessee; Hydraulic Model Investigation," Technical Memoranda 2-381, Vicksburg, Miss.

A13. US Army Engineer Waterways Experiment Station. 1954. "Spillway for New Cumberland Dam, Ohio River, West Virginia; Hydraulic Model Investigation," Technical Memoranda 2-386, Vicksburg, Miss.

A14. US Army Engineer Waterways Experiment Station. 1955. "Spillway for Gavins Point Dam, Missouri River, Nebraska; Hydraulic Model Investigation," Technical Memoranda 2-404, Vicksburg, Miss.

A15. US Army Engineer Waterways Experiment Station. 1956. "Old River Low-Sill Control Structure; Hydraulic Model Investigation: Report 1, Downpull Forces on Vertical-Lift Gates," Technical Report 2-447/1, Vicksburg, Miss.

A16. US Army Engineer Waterways Experiment Station. 1957. "Old River Low-Sill Control Structure; Hydraulic Model Investigation: Report 2, Studies of Riprap Placement Plans," Technical Report 2-447/2, Vicksburg, Miss.

A17. US Army Engineer Waterways Experiment Station. 1959. "Old River Low-Sill Control Structure; Hydraulic Model Investigation: Report 3, Study of Over-all Performance," Technical Report 2-447/3, Vicksburg, Miss.

A18. US Army Engineer Waterways Experiment Station. 1958. "Stilling Basin for Warrior Dam, Warrior River, Alabama; Hydraulic Model Investigation," Technical Report 2-485, Vicksburg, Miss.

A19. US Army Engineer Waterways Experiment Station. 1959. "Old River Overbank Structure, Forces on Panel Gates; Hydraulic Model Investigation," Technical Report 2-491, Vicksburg, Miss.

A20. US Army Engineer Waterways Experiment Station. 1959. "Old River Project; Rock-Fill Initial Closure Dam; Hydraulic Model Investigation," Technical Report 2-496, Vicksburg, Miss.

A21. US Army Engineer Waterways Experiment Station. 1959. "Walter F. George Lock and Dam, Chattahoochee River, Alabama and Georgia; Hydraulic Model Investigation," Technical Report 2-519, Vicksburg, Miss.

A22. US Army Engineer Waterways Experiment Station. 1960. "Spillways and Stilling Basins, Jackson Dam, Tombigbee River, Alabama; Hydraulic Model Investigation," Technical Report 2-531, Vicksburg, Miss.

A23. US Army Engineer Waterways Experiment Station. 1960. "Dardanelle Lock and Dam, Arkansas River, Arkansas; Hydraulic Model Investigation," Technical Report 2-558, Vicksburg, Miss.

A24. US Army Engineer Waterways Experiment Station. 1961. "Spillway for Markland Locks and Dam, Ohio River, Kentucky and Indiana; Hydraulic Model Investigation," Technical Report 2-566, Vicksburg, Miss.

A25. US Army Engineer Waterways Experiment Station. 1961. "Spillway, Greenup Locks and Dam, Ohio River, Kentucky and Indiana; Hydraulic Model Investigation," Technical Report 2-572, Vicksburg, Miss.

A26. US Army Engineer Waterways Experiment Station. 1961. "Spillway and Stilling Basin, Columbia Dam, Chattahoochee River, Alabama and Georgia; Hydraulic Model Investigation, " Technical Report 2-578, Vicksburg, Miss.

A27. US Army Engineer Waterways Experiment Station. 1961. "Spillway Stilling Basins for Maxwell and Opekiska Locks and Dams, Monongahela River, Pennsylvania and West Virginia; Hydraulic Model Investigation," Technical Report 2-579, Vicksburg, Miss.

A28. US Army Engineer Waterways Experiment Station. 1961. "Spillway for New Cumberland Locks and Dam (Final Design), Ohio River, West Virginia; Hydraulic Model Investigation," Technical Report 2-585, Vicksburg, Miss.

A29. US Army Engineer Waterways Experiment Station. 1961. "Spillway Stilling Basin for Pike Island Locks and Dam, Ohio River, Ohio and West Virginia; Hydraulic Model Investigation," Technical Report 2-586, Vicksburg, Miss.

A30. US Army Engineer Waterways Experiment Station. 1963. "Typical Spillway Structure for Central and Southern Florida Water-Control Project; Hydraulic Model Investigation, " Technical Report 2-633, Vicksburg, Miss.

A31. US Army Engineer Waterways Experiment Station. 1964. "Spillway, Millers Ferry Lock and Dam, Alabama River, Alabama; Hydraulic Model Investigation," Technical Report 2-643, Vicksburg, Miss.

A32. US Army Engineer Waterways Experiment Station. 1964. "Spillway for Proctor Dam, Leon River, Texas; Hydraulic Model Investigation," Technical Report 2-645, Vicksburg, Miss.

A33. US Army Engineer Waterways Experiment Station. 1964. "Stability of Riprap and Discharge Characteristics, Overflow Embankments, Arkansas River, Arkansas; Hydraulic Model Investigation, Technical Report 2-650, Vicksburg, Miss.

A34. Grace, J. L. 1964. "Spillway for Typical Low-Head Navigation Dam, Arkansas River, Arkansas; Hydraulic Model Investigation," Appendix A: "Gate Sills and Stilling Basins for Locks and Dams Nos. 5 and 7; Hydraulic Model Investigation," by R. S. Cummins and J. L. Grace. Technical Report 2-655, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A35. Melsheimer, E. S., and Rooke, A. D. 1964. "Spillway for Oahe Dam, Missouri River, South Dakota; Hydraulic Model Investigation," Technical Report 2-657, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A36. Pickering, G. A. 1965. "Spillway for Belleville Locks and Dam, Ohio River, Ohio and West Virginia; Hydraulic Model Investigation," Technical Report 2-687, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A37. Murphy, T. E., and Cummins, R. S. 1965. "Tests of Structure Orientation, Spillway, and Lock Emergency Gate, Barkley Lock and Dam, Cumberland River, Kentucky; Hydraulic Model Investigation," Technical Report 2-689, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A38. Grace, J. L., and Pickering, G. A. 1965. "Spillway for Cannelton Locks and Dam, Ohio River," Technical Report 2-710, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A39. Pickering, G. A. 1966. "Spillway, Hannibal Locks and Dam, Ohio River, Ohio and West Virginia; Hydraulic Model Investigation," Technical Report 2-731, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A40. Melsheimer, E. S., and Murphy, T. E. 1966. "Spillway and Navigation Conditions, Holt Lock and Dam, Warrior River, Alabama; Hydraulic Model Investigation," Technical Report 2-745, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A41. Fletcher, B. P., and Grace, J. L. 1969. "Spillway for Hugo Dam, Kiamichi River, Oklahoma; Hydraulic Model Investigation," Technical Report H-69-15, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A42. Fletcher, B. P. 1970. "Spillway for Copan Dam, Little Caney River, Oklahoma; Hydraulic Model Investigation," Technical Report H-70-9, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A43. Melsheimer, E. S. 1970. "Spillway for Oakley Dam, Sangamon River, Illinois; Hydraulic Model Investigation," Appendix A: "Type 2 (Revised) Spillway; Hydraulic Model Investigation," Technical Report H-70-13, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A44. Pickering, G. A. 1971. "Spillway Gate Vibrations on Arkansas River Dams, Arkansas and Oklahoma; Hydraulic Model Investigation," Technical Report H-71-5, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A45. Oswalt, N. R., and Pickering, G. A. 1973. "Spillway for Lock and Dam 26, Mississippi River, Missouri and Illinois; Hydraulic Model Investigation," Technical Report H-73-15, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A46. Oswalt, N. R. 1974. "Spillway for Aliceville Lock and Dam, Tombigbee River, Alabama; Hydraulic Model Investigation," Technical Report H-74-10, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A47. Oswalt, N. R., and Pickering, G. A. 1974. "Spillway for Columbus Lock and Dam, Tombigbee River, Alabama; Hydraulic Model Investigation," Technical Report H-74-13, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A48. Neilson, F. M., and Tool, A. R. 1976,. "Prototype Tests, Old River Low-Sill Control Structure, April 1973-June 1975," Technical Report H-76-15, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A49. Rothwell, E. D., and Grace, J. L., Jr. 1977. "Old River Existing Low-Sill Control Structure, Louisiana; Hydraulic Model Investigation,'* Technical Report H-77-2, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A50. Pugh, C. A. 1977. "Spillway Vibration, Pressure, and Velocity Measurements, Ozark Lock and Dam, Arkansas River, Arkansas," Technical Report H-77-6, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A51. Oswalt, N. R. 1977. "Red River Waterway, La., Tex., Ark., and Okla., Mississippi River to Shreveport, La., Reach, Lock and Dam No. 1; Hydraulic Model Investigation, Technical Report H-77-13, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A52. Fletcher, B. P. 1978. "Typical Tennessee-Tombigbee Canal Section Spillways A and B; Hydraulic Model Investigation," Technical Report H-78-21, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A53. Hart, E. D., and Hite, J. E., Jr. 1979. "Prototype Gate Vibration Tests, Barkley Dam, Cumberland River, Kentucky," Technical Report HL-79-8, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A54. Rothwell, E. D., Oswalt, N. R., and Maynord, S. T. 1981. "Grays Landing Spillway and Stilling Basin, Monongahela River, Pennsylvania; Hydraulic Model Investigation, " Technical Report HL-81-13, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A55. Hite, J. E., Jr., and Pickering, G. A. 1983. "Barkley Dam Spillway Tainter Gate and Emergency Bulkheads, Cumberland River, Kentucky; Hydraulic Model Investigation, " Technical Report HL-83-12, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A56. US Army Engineer Waterways Experiment Station. 1956. "Cavitation at Baffle Piers ," Miscellaneous Paper 2-154, Vicksburg, Miss.

A57. US Army Engineer Waterways Experiment Station. 1961. "Swellhead Effect of Navigation Structures, " Miscellaneous Paper 2-468, Vicksburg, Miss.

A58. US Army Engineer Waterways Experiment Station. 1962. "Fluctuation of Control Gates, " Miscellaneous Paper 2-484, Vicksburg, Miss.

A59. US Army Engineer Waterways Experiment Station. 1963. "Riprap Requirements for Overflow Embankments," Miscellaneous Paper 2-552, Vicksburg, Miss.

A60. US Army Engineer Waterways Experiment Station. 1963. "Discharge Rating Curves for Vertical Lift Gates on Spillway Crests," Miscellaneous Paper 2-606, Vicksburg, Miss.

A61. US Army Engineer Waterways Experiment Station. 1964. "An Investigation of Spillway Bucket and Toe Curve Pressures," Miscellaneous Paper 2-625, Vicksburg, Miss.

A62. Grace, J. L. 1966. "Adequacy of Submerged Rock Weirs to Withstand Propeller Wash, St. Clair River, Michigan; Hydraulic Model Investigation," Miscellaneous Paper 2-821, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A63. Oswalt, N. R. 1968. "Riprap Protection for Subimpoundment Dams, Rend Lake Reservoir, Illinois; Hydraulic Model Investigation," Miscellaneous Paper H-68-1, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A64. Basco, D. R. 1969. "Trends in Baffled, Hydraulic Jump, Stilling Basin Designs of the Corps of Engineers Since 1947," Miscellaneous Paper H-69-1, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A65. Basco, D. R. 1970. "An Experimental Study of Drag Force and Other Performance Criteria of Baffle Blocks in Hydraulic Jumps," Miscellaneous Paper H-70-4, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A66. Murphy, T. E. 1971. "Control of Scour at Hydraulic Structures," Miscellaneous Paper H-71-5, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A67. Cox, R. G. 1972. "Pertinent Data on Spillway Tainter Gates for Corps of Engineers Projects," Miscellaneous Paper H-72-7, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A68. Cox, R. G. 1973. "Spillway Water-Surface Profiles," Miscellaneous Paper H-73-4, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A69. Murphy, T. E. 1973. "Spillway Crest Design," Miscellaneous Paper H-73-5, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A70. Neilson, F. M. 1976. "Convex Chutes in Converging Supercritical Flow,*' Miscellaneous Paper H-76-19, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A71. Copeland, R. R. 1980. "Old River Overbank Structure Outlet Modifications; Hydraulic Model Investigation," Miscellaneous Paper HL-80-5, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A72. Lecher, F. A. 1971. "Some Characteristics of Pressure Fluctuations on Low-Ogee Crest Spillways Relevant to Flow-Induced Structural Vibrations," Contract Report H-71-1, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

A73. US Army Engineer Waterways Experiment Station. 1931. "Hydraulic Studies of Proposed Dam No. 37, Ohio River, Paper D, Vicksburg, Miss.

A74. US Army Engineer Waterways Experiment Station. 1931. "Experiments to Determine the Erosive Effects of Floodwaters on Railroad Embankments," Paper R, Vicksburg, Miss.

A75. US Army Engineer Waterways Experiment Station. 1933. "Model Studies of Spillways for St. Lucie Canal, Martin County, Florida," Paper 14, Vicksburg, Miss.

A76 US Army Engineer Waterways Experiment Station. 1934. "Experiments to Determine the Backwater Effects of Submerged Sills in the St. Clair River, Paper 16, Vicksburg, Miss.

A77. Nelson, M. E. 1932. "Laboratory Tests on Hydraulic Models of the Hastings Dam," Report No. 1, US Army Engineer District, St. Paul, St. Paul, Minn.

A78. US Army Engineer District, St. Paul. 1934. "Preliminary Report, Laboratory Tests on Hydraulic Models of Sand Dams, 1/5th Size Model," Hydraulic Laboratory Report No. 2, St. Paul, Minn.

A79. US Army Engineer District, St. Paul. 1932. "Laboratory Tests on Hydraulic Model of Proposed Lock and Dam No. 2, Kiskiminitas River," Hydraulic Laboratory Report No. 3, St. Paul, Minn.

A80. US Army Engineer District, St. Paul. 1932. "Laboratory Tests on Hydraulic Model of Beartrap Dam, Ohio River Type," Hydraulic Laboratory Report No. 4, St. Paul, Minn.

A81. US Army Engineer District, St. Paul. 1932. "Laboratory Tests on Hydraulic Models of Marmet Lock and Dam, Kanawha River," Hydraulic Laboratory Report No. 5, St. Paul, Minn.

A82. US Army Engineer District, St. Paul. 1934. "Final Report on Laboratory Tests on Hydraulic Models of Lock and Dam No. 15, Mississippi River," Hydraulic Laboratory Report No. 7, St. Paul, Minn.

A83. US Army Engineer District, St. Paul. 1935. "Final Report on Laboratory Tests on Hydraulic Models of Lock and Dam No. 15, Mississippi River, Appendix A: Comparison of Model to Prototype," Hydraulic Laboratory Report No. 9, St. Paul, Minn.

A84. US Army Engineer District, St. Paul. 1936. "Final Report, Laboratory Tests on Hydraulic Models of New Lock and Dam No. 4, Monongahela River, Charleroi, Pennsylvania," Hydraulic Laboratory Report No. 12, St. Paul, Minn.

A85. US Army Engineer District, St. Paul. 1936. "Final Report, Laboratory Tests on Hydraulic Models to Determine Roller Gate Coefficients," Hydraulic Laboratory Report No. 13, St. Paul, Minn.

A86. US Army Engineer District, St. Paul. 1936. "Final Report, Laboratory Tests on Hydraulic Model of Montgomery Island Lock and Dam, Ohio River, Near Industry, Ohio," Hydraulic Laboratory Report No. 14, St. Paul, Minn.

A87. US Army Engineer District, St. Paul. 1936. "Final Report, Laboratory Tests on Hydraulic Models of Winfield Lock and Dam, Kanawha River, Winfield, West Virginia," Hydraulic Laboratory Report No. 15, St. Paul, Minn.

A88. US Army Engineer District, St. Paul. 1937. "Final Report, Laboratory Tests on Hydraulic Models to Determine Roller Gate Coefficients for Dams No. 5, 5A, and 8, Mississippi River, Hydraulic Laboratory Report No. 17, St. Paul, Minn.

A89. US Army Engineer District, St. Paul. 1937. "Final Report, Laboratory Tests on Hydraulic Models of Lock and Dam No. 26, Mississippi River, Alton, Illinois," Hydraulic Laboratory Report No. 20, St. Paul, Minn.

A90. US Army Engineer District, St. Paul. 1937. "Final Report, Laboratory Tests on Hydraulic Models of Peoria and La Grange Dams, Illinois River, Peoria and La Grange, Illinois, Hydraulic Laboratory Report No. 23, St. Paul, Minn.

A91. US Army Engineer District, St. Paul. 1937. "Final Report, Laboratory Tests on Hydraulic Model of Lock and Dam No. 20, Mississippi River, Canton, Missouri," Hydraulic Laboratory Report No. 24, St. Paul, Minn.

A92. US Army Engineer District, St. Paul. 1938. "Final Report, Laboratory Tests on Hydraulic Models of Lock and Dam No. 7, Mississippi River, Onalaska Spillway," Hydraulic Laboratory, Report No. 29, St. Paul, Minn.

A93. US Army Engineer District, St. Paul. 1938. "Final Report, Laboratory Tests on Hydraulic Models to Determine Discharge Through Chanoine Wicket Dams," Hydraulic Laboratory Report No. 30, St. Paul, Minn.

A94. US Army Engineer District, St. Paul. 1938. "Final Report, Laboratory Tests on Hydraulic Models to Determine Tainter Gate Coefficients for Dams No. 5, 5A, and 8, Mississippi River," Hydraulic Laboratory Report No. 31, St. Paul, Minn.

A95. US Army Engineer District, St. Paul. 1939. "Final Report, Laboratory Tests on Hydraulic Model of Lock and Dam No. 22, Mississippi River, Hannibal, Missouri, Hydraulic Laboratory Report No. 33, St, Paul, Minn.

A96. US Army Engineer District, St. Paul. 1939. "Final Report, Laboratory Tests on Hydraulic Models of Roller Gate Stilling Basins," Hydraulic Laboratory Report No. 36, St. Paul, Minn.

A97. US Army Engineer District, St. Paul. 1939. "Final Report, Laboratory Tests on Hydraulic Models of Submergible Tainter Gates," Hydraulic Laboratory Report No. 37, St. Paul, Minn.

A98. US Army Engineer District, St. Paul. 1940. "Laboratory Tests on Hydraulic Model to Determine Roller Gate Coefficients, Appendix A," Hydraulic Laboratory Report No. 39, St. Paul, Minn.

A99. US Army Engineer District, St. Paul. 1948. "Final Report, Laboratory Tests on Hydraulic Models of Lock and Dam No. 4, Mississippi River," Hydraulic Laboratory Report No. 53, St. Paul, Minn.

B01. US Army Engineer District, St. Paul. 1959. "Final Report, Laboratory Tests on Hydraulic Model of Spillway Apron for Dam No. 1, Mississippi River," Hydraulic Laboratory Report No. 63, St. Paul, Minn.

B02. US Army Engineer District, St. Paul. 1960. "Lower Lock and Dam Tainter Gates, St. Anthony Falls Upper Harbor Project, Mississippi River, Minneapolis, Minnesota; Hydraulic Model Investigation," Hydraulic Laboratory Report No. 69, St. Paul, Minn.

B03. US Army Engineer District, St. Paul. 1963. "A Study of Water Pressures on Roller Gates; Hydraulic Model Investigation," Hydraulic Laboratory Report No. 77, St. Paul, Minn.

B04. Bonneville Hydraulic Laboratory. 1941. "Spillway Pressure Head Investigation, Bonneville Spillway Dam, Bonneville, Oregon," Technical Report No. 3-1, Bonneville, Ore.

B05. Bonneville Hydraulic Laboratory. 1956. "McNary Dam, Columbia River, Oregon & Washington; Hydraulic Model Investigation," Technical Report No. 20-1, Bonneville, Ore.

B06. Bonneville Hydraulic Laboratory. 1961. "Spillway and Stilling Basin for McNary Dam, Columbia River, Oregon & Washington; Hydraulic Model Investigations," Technical Report No. 21-1, Bonneville, Ore.

B07. Perkins, L. Z. 1973. "Ice Harbor Dam, Snake River, Washington; Hydraulic Model Investigation," Technical Report No. 22-1, Division Hydraulic Laboratory, US Army Engineer Division, North Pacific, Bonneville, Ore.

B08. US Army Engineer Division, North Pacific. 1965. "Spillway and Stilling Basin for Ice Harbor Dam, Snake River, Washington; Hydraulic Model Investigation," Technical Report No. 31-1, Division Hydraulic Laboratory, Bonneville, Ore.

B09. US Army Engineer Division, North Pacific. 1964. "Spillway and Stilling Basin for the Dalles Dam, Columbia River, Oregon and Washington," Technical Report No. 55-1, Division Hydraulic Laboratory, Bonneville, Ore.

B10. Bonneville Hydraulic Laboratory. 1958. "Revised Stilling Basin, Bonneville Dam, Columbia River, Oregon & Washington; Hydraulic Model Investigation," Technical Report No. 65-1, Bonneville, Ore.

B11. US Army Engineer Division, North Pacific. 1974. "Spillway and Stilling Basin, John Day Dam, Columbia River, Oregon and Washington; Hydraulic Model Investigation," Technical Report No. 97-1, Division Hydraulic Laboratory, Bonneville, Ore.

B12. US Army Engineer Division, North Pacific. 1967. "Spillway and Stilling Basin, Lower Monumental Dam, Snake River, Washington; Hydraulic Model Investigation," Technical Report No. 104-1, Division Hydraulic Laboratory, Bonneville, Ore.

B13. Johnson, R. L., and Perkins, L. Z. 1975. "Spillway for Little Goose Dam, Snake River, Washington; Hydraulic Model Investigation," Technical Report No. 114-1, Division Hydraulic Laboratory, US Army Engineer Division, North Pacific, Bonneville, Ore.

Bl4. Edmister, T. D., and Smith, P. M. 1975. "Spillway Gate Modification, Bonneville Dam, Columbia River, Oregon and Washington; Hydraulic Model Investigation," Technical Report No. 136-1, Division Hydraulic Laboratory, US Army Engineer Division, North Pacific, Bonneville, Ore.

B15. Carnegie Institute of Technology. 1937. "Report on Hydraulic Model Tests of Lock and Dam No. 9, Allegheny River Near Rimerton, Pennsylvania," Hydraulic Research Laboratory, Pittsburgh, Pa.

B16. Carnegie Institute of Technology. 1941. "A Report on Hydraulic Model Tests of Power-Operated Wicket in Connection with Ohio River Replacement Study," Hydraulic Research Laboratory, Pittsburgh, Pa.

B17. Carnegie Institute of Technology. 1942. "A Report on Model Studies on Cavitation, Bluestone Dam, New River, Hinton, W. Va.," Hydraulic Research Laboratory, Pittsburgh, Pa.

B18. Carnegie Institute of Technology. 1943. "Report on Model Studies on Cavitation, Bonneville Dam, Bonneville, Oregon," Hydraulic Research Laboratory, Pittsburgh, Pa.

B19. Thomas, H. A., and Schuleen, E. P. 1936." Warrior River, Tuscaloosa Lock and Dam, Report on Hydraulic Model Tests," Carnegie Institute of Technology and US Engineer Office, Pittsburgh, Pa.

B20. US Army Engineer District, Huntington. 1937. "Final Report, Laboratory Tests on Hydraulic Models of Gallipolis Locks and Dam, Ohio River," Huntington, W. Va.

B21. US Army Engineer District, Huntington. 1939. "Report on Field Checks on Model Tests at Gallipolis Locks & Dam, Ohio River and London Locks & Dam, Kanawha River," Huntington, W. Va.

APPENDIX D

TYPICAL SPILLWAY OPTIMIZATION STUDY

(Red River, Louisiana)

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

2. DESIGN GUIDANCE FOR NAVIGATION DAM STRUCTURES.

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

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

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

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

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

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

3. FLOWAGE EASEMENTS.

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

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

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

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

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

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

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

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

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

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

6. CONCLUSIONS AND RECOMMENDATIONS.

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

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

TABLE D-1

Spillway Arrangements That Would Raise 178,000 cfs Above Preproject

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

TABLE D-2

Spillway Arrangements That Would Raise the PDF Above Preproject

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

TABLE D-3

Comparative Costs

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

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

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

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

U.S. GOVERNMENT PRINTING OFFICE: 1987 - 720-724/60172