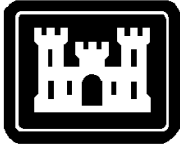| CECW-ED Engineer Manual 1110-2-2607	Department of the Army U.S. Army Corps of Engineers Washington, DC 20314-1000	EM 1110-2-2607 31 July 1995
	Engineering and Design PLANNING AND DESIGN OF NAVIGATION DAMS	
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US Army Corps
of Engineers

ENGINEERING AND DESIGN

Planning and Design of Navigation Dams

ENGINEER MANUAL

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Engineering and Design
PLANNING AND DESIGN OF NAVIGATION DAMS

1. Purpose. This manual is issued for guidance of individuals and elements within the Corps of Engineers engaged in the structural planning, layout, and design of navigation dams for civil works projects. The structural design of gates is not covered in this manual.

2. Applicability. This manual applies to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities (FOA) having responsibilities for the design and construction of civil works projects.

FOR THE COMMANDER:

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Chief of Staff

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U.S. Army Corps of Engineers
CECW-ED   **Washington, DC 20314-1000**

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Engineering and Design
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Chapter 1
Introduction

1-1. Purpose

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1-2. Applicability

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1-3. References

Required and related publications are listed in Appendix A.

1-4. Scope

This manual provides guidance for the planning and design of navigation dams and appurtenant structural features. The appurtenant features include embankments, dam piers, stilling basins, training walls, separation walls, galleries, sills, service bridges, and machinery houses. This manual is structured so that Chapters 2 through 4 contain layout and planning information; Chapter 5 provides detailed descriptions of types of dam structures; Chapter 6 discusses maintenance and emergency closure facilities; Chapter 7 discusses appurtenant seepage control features; Chapter 8 discusses appurtenant channel protection features; Chapters 9 through 11 present detailed requirements for stability and structural analysis and design; Chapter 12 discusses concrete and joint materials, waterstops, etc.; and Chapter 13 discusses design consideration for miscellaneous dam features. Appendix A covers references; Appendix B contains a design check list; and Appendix C describes lessons learned from case histories.

1-5. Supersession

This EM supersedes EM 1110-2-2607, dated 1 July 1958.

Chapter 2
General Considerations for Project Planning

2-1. General

This chapter provides an overview of the engineering, policy, and planning guidance applicable to developing a project plan for navigation improvements associated with the planning, design, construction, and major rehabilitation of navigation dams. Although there are a number of similarities among low-, medium-, and high-head lock and dam projects, care should be taken in use of this manual to ensure that guidance which applies only to low-head projects is not misconstrued as applicable to high-head projects. Because a navigation dam is usually planned concurrently with a companion navigation lock, the planning effort usually considers both types of navigational structures in the same studies. Therefore, the intent is for EM 1110-2-2602 to be used in conjunction with this manual as that manual covers many of the items mentioned below in more detail.

2-2. Project Team

The planning, engineering layout, and design of navigation dams as part of the overall development of a project plan for navigation projects comprise a complex, multidisciplinary planning and engineering effort. This effort involves the contributions from many public and private interests including local, state, and federal agencies; planners; design, operations, and construction engineers; architects; and natural habitat biologists. The team will either include or derive considerable input from budgetary, legal, and contracting specialists.

2-3. Project Formulation and Development Processes

The Corps of Engineers' involvement in these processes for navigation dams, i.e., civil works reconnaissance studies, feasibility reports, and preconstruction engineering and design (PED), is very similar to that for navigation locks, as covered in EM 1110-2-2602. Also, guidance for developing cost estimates discussed in that manual is applicable to the navigation dam. Since completion of the PED work is frequently scheduled to be at different times for the lock and the dam due to limited resources, it is important to carefully coordinate the details for these structural entities so that a quality end product is completed on time and within budget.

2-4. Legal Environment

Legal issues which must be considered in project development include, but are not limited to, method of funding, navigation servitude, environmental issues, historic properties, archeological concerns, and the National Historic Preservation Act. These issues will frequently be controlling factors in the overall project development process.

2-5. Project Components and General Feature Requirements

a. General layout. A navigation dam is similar to most other dams in that its intended purpose is to impound water. However, it is usually designed so that the water surface upstream of the dam is of such elevation that there will be sufficient depth for navigating a relatively long distance upstream from the dam without having to go through another lock or to dredge excessively. However, there are a number of factors--basically related to costs and impacts on existing developments and on the environment--which place limits on the upper pool which will be maintained by the dam. Except possibly in the case of impoundment for water supply, hydroelectric power, or flood control, the dam is designed to allow the same amount of flow through the waterway as existed prior to construction of the dam. The height, or lift, of a dam is the difference in elevations of the upstream pool formed by the dam and the minimum pool or natural stream surface below the dam. Passage of vessels between the upper and lower water levels is accomplished by a navigation lock.

b. Operational requirements. Because the primary purposes of a navigation dam are to impound water and to regulate stream flows so waterway traffic can lock through under almost all pool conditions, it is extremely important that the project structures be laid out to allow safe, timely, and efficient transit of the lock. The configuration of the upstream and downstream reaches of the stream and the currents and velocities of the stream caused by spillway discharges are critical factors in the proper layout of the project--particularly for layout of the upstream and downstream approaches to the lock.

c. Basic project components for low- to medium-head dams. Lock and dam projects can be single-purpose and only consider navigation, or may be developed for multipurpose uses. The basic components of a low- to medium-head navigation project (a low- to medium-head dam is generally considered to be one with a normal lift or differential between upper and lower pool levels of

50 ft or less) could include one or more locks and several of the following dam features:

(1) Gated dam and spillway or sill with stilling basin and training and grade separation walls. The gates will typically be of the tainter type. However, in some instances, wicket or hinged-crest gates will be used in conjunction with or in lieu of tainter gates.

(2) Overflow embankment or weir.

(3) Navigation pass, which will normally serve as the above-mentioned weir, may include hinged-crest or wicket gates.

(4) Nonoverflow embankment or concrete or cellular walls (lock and dam separation and/or dam abutment).

(5) Gate operating machinery such as hoists and hydraulic systems.

(6) Maintenance and/or emergency bulkheads for the gates.

(7) Service bridge--typically limited to the tainter-gated portion of the dam.

(8) Drainage and grouting galleries.

(9) Seepage cutoff walls or other seepage control measures.

(10) Channel armoring usually consisting of derrick stone and riprap and, in some cases, concrete paving.

(11) Buildings which will normally be used to facilitate both lock and dam operation (visitors, recreation, administration, maintenance, storage, etc.).

(12) Electrical power generation using "run-of-the-river" flow--unless additional impoundment to extend the generating period is feasible. Currently, inclusion of power generation is a development of (and is funded by) entities other than the federal government. However, the generating component has to conform to navigation requirements and meet the stability and safety requirements of HQUSACE.

d. Project components for high-head dams. High-head dams (with a greater than 50-ft differential between upper and lower pools) will contain most of the features mentioned above for low- to medium-head dams, except

they do not have navigation passes. (There will always be a permanent differential in upstream and downstream pools.) These high-head dams are not likely to contain hinged-crest or wicket gate sections. The following are important characteristics of high-head dams:

(1) Vertical-lift gates have typically been used in lieu of tainter gates because the structural dimensions for the lift gates and piers are less than required for tainter gates. However, tainter gates with supplemental sluice (vertical-lift) gates are likely to be used more extensively in the future as the procedures for operating the tainter gates are simple and require minimal attendant labor--unlike the vertical-lift gate.

(2) Power-generating facilities will normally be incorporated because there is typically a sufficient impoundment above the conservation pool to make power generation economically feasible. Also, these facilities may be subject to private development and operation--depending on the legal environment at the time of construction.

(3) Frequently, fish ladders will be provided if the species of fish in the dam's locality are migratory in nature, such as those in the northwestern United States.

e. Multipurpose project components. The navigation project with multiple-purpose functions should accommodate each purpose as much as is economically justified and technically feasible with priority of purposes taken into account. Common multipurpose components are:

(1) Navigation lock(s) and dam.

(2) Flood control capability.

(3) A powerhouse.

(4) Fish passage facilities.

(5) Recreation facilities (boating and other water sports, enhanced fisheries, picnic facilities, etc.).

(6) Water supply intakes for municipalities and/or irrigation.

(7) Features to enhance water quality downstream of the dam, e.g., low flow controls and multilevel outlets.

(8) Water conservation.

2-6. Examples of Navigation Projects and Components of Dams

Except for the John Day Dam mentioned below, all of the following are features of low- to medium-head projects:

a. Plates 1 and 2 are provided to show a perspective and a section of a typical lock and dam on the Red River Waterway and the various structural features used in this dam. Primary control of flow is regulated by tainter gates. The crest gate spillway is not provided as a navigation pass but primarily to supplement the flow capacity of the tainter gate spillway during high river stages. The crest gates are lowered during high water to increase flow capacity and are used to fine-tune discharge rates when the river is in pool.

b. Plate 3 shows the proposed Olmsted Locks and Dam. This dam's primary feature is the wide navigation pass which is to be used by river traffic during a high percentage of the year. During low-water stages the upper pool will be maintained by wicket gates, and navigation traffic will pass through the lock. Typical sections through the navigable pass monoliths are shown in Plate 4.

c. The Melvin Price Locks and Dam project is presented in plan in Plate 5. This example includes nine identical, 110-ft-wide tainter gate bays with two of the nine used to separate the main and auxiliary locks. The overflow section on the Missouri side is not designed to function as a navigation pass. Plate 6 presents closeup views of the tainter gate bay features.

d. The Smithland Locks and Dam project is presented in Plates 7-9. This project is similar to the Melvin Price project mentioned in paragraph 2-6*c* above. It has 11 identical, 110-ft-wide tainter gates and a non-navigable fixed weir.

e. Examples of usage of earth-fill sections as a damming surface or element are shown in Plates 10 and 11.

f. Features of the John Day Dam, an example of a high-head dam with multiple-purpose functions, are presented in Plates 12 and 13. These features include tainter gate bays, a lock structure, a powerhouse section, and a fish ladder.

2-7. Existing Conditions

Much of the following will not be applicable to major rehabilitation projects; however, the opposite is true for new projects. Once the need for a navigation dam is identified, a careful assessment must be made of the natural physical characteristics of a stream and its valley, as well as the conceivable dam sites. Various existing site conditions can have profound effects on cost, operational feasibility, and acceptability to numerous entities and interests.

a. Site conditions and restrictions. In long-settled and developed regions, existing maps, geological surveys, and hydrometeorological records may provide sufficient data for preliminary design purposes. In other regions, extensive field surveys and research of hydrological and climatological records will be required before the project can be designed. This subject is discussed more extensively in EM 1110-2-2602.

b. Climate.

(1) Range of temperature. Temperature extremes of either heat or cold will influence the general dam design and the detailed design of operating components and structural features. In cases of extreme cold, the possibility of ice formation must be considered. Structural design should include allowance for ice thrust on the structural features exposed to the pool surfaces and on gates, as well as the impact and abrasion of running ice. Floodway openings must be designed to pass large volumes of ice to minimize the danger of ice jams forming. In extremely cold climates, heating systems may be required for winter operation of gates.

(2) Humidity. The degree of humidity inherent to location on a water course must be considered in design of electrical services, machinery, and corrosion protection. The occurrence of frequent or prolonged fog or a tropical combination of heat and humidity may present major problems in design and maintenance of electrical machinery and a structure's metal parts.

(3) Climatological records. In the United States, climatological data such as precipitation, evaporation, wind speed and direction, and temperature are archived in various formats by the National Oceanic and Atmospheric

Administration (NOAA), a unit of the U.S. Department of Commerce.

c. Topography. The plan for a navigation dam structure should conform to the topographical features of the project area. If the required information is not already available, maps should be prepared to show the pertinent data based on surveys conducted for this purpose. Data required for planning and design of the project include but are not limited to the following:

(1) Information in regard to populated areas (location, elevation, and other items) which will indicate possible effects from project construction.

(2) Locations of railroads, highways, power lines, natural gas pipe lines, flood protection projects, levees, sewer outlets, water-supply intakes, pumping stations, and input from the owners of those features which may be affected by the proposed project.

(3) Locations of fishing and hunting preserves and input from the owners of those features which may be affected by the proposed project.

(4) Locations and pertinent data on bridges, dams, dikes, wharves, pleasure resorts, and all other features that might be affected by the project.

(5) Channel soundings, high and low water marks, gage and historical river gage data.

d. Hydrologic and hydraulic.

(1) Hydrologic studies. A watershed hydrology study is one of the first needs in developing a navigable waterway. The hydrologic conditions along the waterway length will determine required lift needed for a dam to establish reliable navigation. Hydrologic studies for a river basin identify the discharge frequencies and duration a dam structure (located at any particular point within the basin) must be designed to accommodate. Minimum, normal, and maximum discharges are all significant to the dam design.

(2) Hydraulic studies. Hydraulic studies for navigation dam design generally cover two distinct phases. The first phase establishes the stage-discharge relationship and its effect on the entire area affected by the proposed project under both existing and postproject conditions. The second phase of hydraulic studies involves the design of dams and other structures (i.e., their type, shape, size, and siting to ensure satisfactory hydraulic performance). A

more detailed coverage of all aspects of hydrology and hydraulics, as these items relate to navigation dams, including existing data and record sources, is found in ER 1110-2-1404, ER 1110-2-1458, EM 1110-2-1604, ER 1110-2-1461, and EM 1110-2-1605. For guidelines that cover special hydraulic features of a project, see EP 25-1-1.

e. Geology. To properly evaluate the suitability of a site for location of the navigation dam and lock structures, it is necessary to assemble and evaluate all the available geologic information and perform new core drilling, probings, and soundings. Composition and depth of overburden, quality and type of underlying rock, and quality and type of exposed rock are extremely important factors. Subsequent foundation studies, based on the assembled geologic information, will help determine whether the structure should be founded on rock, soil, or piling. Also, the geology of the stream bed will influence sediment transport and stream-bed stability requirements.

f. Existing land ownership and usage. The consideration of real estate is not limited to the amounts and locations of that needed for the project and the associated costs but must also include the current land uses and the environmental and social issues associated with these uses. Some real estate usage is so sensitive that development of a project based on usage of such "sensitive" real estate would never come to fruition in today's political and legal climates. Real estate requirements for the project features are discussed in more detail in paragraph 2-14.

g. Environmental setting. Information on existing environmental conditions will be necessary to prepare the compliance documents required by existing federal regulations. Early and continuing communication with agencies charged with protection of the environment is essential. A finding of no significant impact (FONSI) is a prerequisite to project development and construction. Careful planning to maintain or enhance the environmental quality and mitigation measures may preclude or set aside the potential negative impacts that would render the project infeasible or not allow its approval. Also, high quality resource management plans plus improved design and operation procedures will help maximize environmental benefits and help attain environmental quality objectives.

2-8. Navigation and Pool Operational Considerations

a. General. In general, the lock features and location prevail in importance to other project features and

purposes. However, the following considerations associated with the dam and approach channels will have considerable bearing on good navigational approach conditions.

b. Channel depth and width. Navigation will be enhanced by providing channel depths and widths for movement and maneuvering of vessels at the desired speeds, eliminating hazardous currents, and providing pools stable enough to allow development of suitable terminal facilities by navigation interests. The efficiency of navigation can be enhanced by including navigation passes for low-head dams and mooring facilities at locations remote from the dam to ensure that unattended tows are not drawn into the dam and do not drift into the path of river traffic. The bases for channel depth, channel width, and lock dimensions are established by study of a number of factors, including types and probable future tonnage of traffic, types and sizes of vessels in general use on connecting waterways, and developments on other waterways which may be indicative of the type and size of vessels likely to use the channel.

c. Control of hazardous currents. The slack-water pools created by dams will reduce current velocities from those existing in the stream's natural state and, in general, will eliminate hazardous rapids. However, new hazards may be created. Vessels entering or leaving the locks will have limited steering control at required low speeds and can be drawn out of control by currents set up by the spillway section of the dam. Approach walls and other protection, for a considerable distance above or below the lock, may be required to hold tows in line. Some restriction on the use of spillway gates adjacent to the lock may be necessary. Maximum velocities and channel depths usually will be found along the outer bank of bends, and even slight curvature will tend to fix the natural deepwater channel closer to the outer bank. A lock aligned with the natural deepwater channel will usually provide the best navigation characteristics; locations in sharp bends and where the lock structure will deflect a substantial part of the flow should be avoided. Model studies and advice of experienced masters and pilots should be considered in preparing a lock and dam layout to avoid hazardous current conditions.

d. Regulation of upper pool.

(1) General. In addition to the flooding impacts of the selected pool, consideration should be given to the pool's operational stability. A navigation dam should ordinarily provide a fixed pool elevation with little stage variation. Dependable minimum upper pool stages

promote navigational reliability, growth in waterway traffic, and simplified development of port facilities. To maintain the upper pool elevation at as near a constant level as possible, gated spillway bays are usually provided in navigation dams so that, by controlled gate operation, both normal and flood flows can be passed downstream through the bays.

(2) Configuration of dam. The regulation of all flows from the impounded pool in the most efficient and nondetrimental manner requires that careful consideration be given to the functional shape, elevations, lengths, and widths of the dam structures.

(3) EM 1110-2-1605 provides information on spillway capacity, spillway shape, spillway gates, stilling basins, pier nose shape, abutments, overflows, and selection of the optimum upper pool elevation.

e. Hinged pool operation.

(1) A principal purpose of the hinged pool operation is to eliminate or minimize impacts which would otherwise result from increasing stages and/or stage frequencies upstream of the dam over those which existed before dam construction. This operation takes into account the flow of water and does not rely on a flat pool operation. Hinged pool operation involves lowering the pool of the dam several feet, usually 2 to 5 ft below normal upper pool level during higher flows where adequate navigation depths are available at the upper end of the pool. When discharge falls the pool is raised to extend the backwater effect above the critical reach to maintain navigation depths.

f. Open river navigation.

(1) General. Where hydraulic conditions allow, it may be desirable to provide a navigable pass across a low damming structure to avoid the tow going through the locking process each time it passes through the navigation facility. Avoiding lockage can provide a substantial time savings for both upbound and downbound tows. 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 are necessary. A navigation pass weir or other section of the dam, with or without crest gates, is necessary to accommodate open river navigation.

(2) Dimensional criteria for a navigable pass. The design must provide sufficient width for safe passage of

tow traffic, including poorly aligned tows. In addition, the pass must have sufficient depth for tows of the authorized draft, including a buffer for overdraft, tow squat, etc. A model study should be considered in the design of a navigable pass. At the present time (1993), the Corps is operating dams with navigable passes on the Ohio, Ouachita, and Black Rivers. Pass widths vary from 200 ft on the Ouachita and Black Rivers to 932 and l,248 ft on the Ohio River. Two new navigable pass dams, Olmsted on the Ohio River and Montgomery Point on the Arkansas River, are in the planning stages. 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 are discussed in Chapter 5. Further material relative to open river navigation can be found in EM 1110-2-1605.

g. Swellhead. The impacts of swellhead (differential head resulting from the flow restriction in the waterway created by the lock and dam) during open river conditions must be considered in the following instances:

(1) When tows are to navigate across a navigation pass, the swellhead in the pass must be small enough to permit the tows to pass through under adequate power to move upstream and otherwise control their alignment. Hydraulic studies must determine the optimum elevation at which the top of the lock wall is to be inundated and the area for flow through (over) the dam components, all to provide flow capacity adequate to minimize swellhead.

(2) Swellhead at most low-head dams, regardless of whether open river navigation is to be provided, can influence the real estate requirements. Swellhead greater than 1 ft may be allowed if open river navigation is not provided and if the costs of associated additional real estate requirements are less than the costs of associated additional flow capacity required to reduce the swellhead.

2-9. Hydraulic Design Considerations

a. General. Much of the basis for hydraulic design has been discussed above under the topics "Existing Conditions" (paragraph 2-7) and "Navigation and Pool Operational Requirements" (paragraph 2-8). Also, the topic "Model Studies" (paragraph 2-10), discussed below, is extremely relevant to hydraulic design. The following items are intended to either reinforce or supplement the content of these referenced topics.

b. Discharge and stages. Hydrologic studies for a river basin identify the discharge which a dam structure must be designed to control in order to satisfy the navigational objectives of the project. The hydraulic studies for navigation dam design cover two phases: establishing the stage-discharge relationship over the entire area affected by the proposed project under both existing and postproject conditions; and designing the dams and other structures, i.e., their type, shape, size, and siting to ensure satisfactory hydraulic performance, including navigation approach conditions and maintenance requirements (i.e., dredging).

(1) Seasonal variations. Project areas subject to periods of low runoff alternating with long periods of high runoff are ideal sites for constructing a navigation dam. However, detailed hydrologic and hydraulic studies must be conducted in all cases to confirm this.

(2) Low flows. A properly functioning navigation project must have sufficient water during low-flow periods to satisfy evaporation losses, seepage from the pool, seepage under the dam, and leakage past the spillway gates, in addition to providing adequate water for lockages. Some projects may also have requirements for water supply, irrigation, hydropower, and environmental needs.

(3) Flood heights. In recent years, the Corps has emphasized providing enough spillway capacity in navigation dams to pass the PMF. However, low-head and medium-head dams of up to 50 ft will usually have an overflow weir, and flood flows may go overbank unless levees are provided. High-head dams of over 50 ft should be provided with enough spillway capacity to pass the PMF.

c. Spillway design.

(1) Low- to medium-head dams. Typically, low- to medium-head navigation dams will be 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 overbank. However, on some low-head projects, extreme floods will overtop the lock walls, and navigation will be directed to cease operations.

(2) High-head dams. Spillways for high-head navigation dams are generally designed to pass the PMF flows. They should also be designed in accordance with other requirements contained in EM 1110-2-1603.

d. Ice conditions. It is necessary to determine the volume and duration of ice conditions at navigation dams so that ice control methods can be developed. Historical

records coupled with site monitoring will be helpful. On a stream where heavy ice formations are present, the spillway bays should be made as wide as practical to aid in passing the ice downstream so that it does not wedge and build up against the structures.

e. References. More detailed descriptions of all the above items, with information on where existing data are available, are contained in EM 1110-2-1605. EM 1110-8-1(FR) and EM 1110-2-1612 provide information and methods of estimating ice situations, growth, and duration using winter air temperatures.

2-10. Model Studies

a. General. Physical model studies of the hydraulic and navigational characteristics of the layout of the dam, the dam spillway, training walls, channels, dikes, slope protection, streambed protection, locks, and lock approaches are an extremely important part of the planning and design for a navigation dam project. The physical model can be either fixed bed or movable bed and should include specific site conditions. Mathematical models, which are likely to be more economical than physical models, are being used to a greater extent as more accurate techniques are developed. The U.S. Army Engineer Waterways Experiment Station (WES) usually conducts these model studies. During preliminary planning stages when alternative layouts and locations are being considered, WES may be able to furnish information based on its experience on other navigation dam projects.

b. Spillway. The navigation dam spillways provide the necessary waterway openings for passing high and low flows to maintain the upper pool level in a range suitable for navigation. In addition to the guidance contained in EM 1110-2-1603, further information for design of the number of spillway bays is required. The width of the bay; the elevation, length, and shape of the spillway crest; pier extensions (separation walls); stilling basin baffles; and end sill can be found in EM 1110-2-1605. This EM contains tabulations of model tests relating to the above-listed items along with comparisons of calculated results versus model study results.

c. Forces on structural components. In some instances, it may be necessary to determine the downpull loading, the buoyancy effect, or possibility of vibration of a structural steel bulkhead or vertical lift gate when the item is being lowered or raised in free-flowing water. A scale model of the structure can be model-tested by WES, and information on loadings, uplift, and vibration tendencies can be obtained.

d. Cofferdams. A movable bed model can be used to examine the different stages of cofferdam construction for a navigation dam project where the cofferdam (e.g., sheet pile cellular type) is located in the existing waterway. The last-stage cofferdam situation can be the most important stage to examine in the model, because this stage will cause an increase in the velocity of the water going through the narrowed river opening. This increased velocity can cause excessive movement of the river bed material all across this location and undermine the river arm of the cofferdam. The modeling results will indicate specifically where the problem locations are. Thus, the cofferdam may be configured with deflectors to shunt the scour away from the cofferdam proper, and the stream bed of the narrowed space where the final dam structure is to be installed can be protected against scour with stone or a weighted lumber mattress.

e. Sediment, debris, and ice handling.

(1) Sediment. A physical movable bed model can be used successfully to determine shoaling and danger points when a stream transports a heavy bed load of sediment. The model cannot predict amounts accurately but can indicate locations where sedimentation is likely to occur if suitable measures are not implemented. Rock dikes, wing dikes, operating procedures, or other preventative measures can be designed into the project.

(2) Debris. The best method for passing debris, especially keeping it away from the upstream lock approach and chamber, can be determined through model studies, and appropriate operating recommendations can be adopted based on the results of these studies.

(3) Ice. Where ice buildup poses a threat to dam structures, physical models of ice control methods can be made at the Ice Engineering Laboratory at the Corps of Engineers Cold Regions Research and Engineering Laboratory (CRREL) in Hanover, NH. EM 1110-2-1612 provides additional information on ice control methods.

f. Stone protection. Physical movable bed models are helpful in determining the locations where stone protection is necessary and in sizing the stone required to protect the river bed and banks from the scouring velocity of the flowing water and wave wash.

2-11. Miscellaneous Engineering Considerations

a. General. This paragraph is not intended to cover all "engineering" requirements but is intended to address some of the engineering topics not otherwise covered in

this chapter. However, inclusion of some items in this paragraph is intended to reinforce information provided elsewhere in this chapter.

b. Summary of principal functional requirements.

(1) The general requirement that dams should cause the least possible delay to movement of traffic may be satisfied by a small number of high nonnavigable dams or a larger number of low dams with navigable passes where practicable. The type of dam selected must be consistent with the flow regimen of the stream and the height of dam permitted by topography and available foundations.

(2) The dam should be sufficiently watertight to maintain the desired pool level at the lowest probable discharge.

(3) Movable spillway gates should be capable of sufficiently rapid and flexible operation to be opened for passage of minor flash floods as well as major floods without excessive fluctuation in pool levels or increase in flood heights, and to be closed in time to hold the desired pool level on a falling flow. Remote operation of gates and individual operating machinery for each gate should be considered.

(4) Floodway openings should be sized to provide for passage of ice floes and drift likely to occur.

(5) The dam should be capable of passing sedimentary material which cannot be permitted to accumulate in the pool.

(6) Design should not induce flooding on the waterway if at all avoidable. If unavoidable, some form of remediation will be required.

c. Location of dam with respect to lock.

(1) General. Unless the navigation lock is in a channel separate from the dam or there is some anomaly in the waterway to dictate a large separation, the dam will typically be located adjacent to the lock and in line with the upstream lock gates. Also, the preferable access to the lock and dam for operation and maintenance is from the lock-side of the waterway, so the dam will typically be located on the side of the lock opposite from the primary operation and maintenance access. Detailed layout requirements, as related to navigation and hydraulics, are provided in EM 1110-2-1611.

(a) Most waterways in the continental United States that have the potential for navigational usage are already developed. Thus, much of the future dam construction is likely to involve rehabilitation or replacement, and it will be done in a manner to facilitate use of portions of the existing navigation features to the maximum extent practical, and will likely involve innovative techniques. The required techniques may not be in total agreement with the preferred layouts discussed in EM 1110-2-1611. For example, the Upper Mississippi River-Illinois Waterway System Navigation Study by the Corps of Engineers North Central Division considers an increase in navigation capacity by building an additional lock chamber through an existing dam with the resulting layout being much different than if the two chambers had been included in the original construction.

(b) From the viewpoint of navigation, siting of the lock is probably more important than siting of the dam because it is desirable to locate the lock within that portion of the channel which will provide the best possible conditions for navigation to approach the lock. Thus, the dam location will usually be controlled by the lock location. Unless flow from the dam is diverted from the downstream lock approach by a physical barrier or there is a large separation between the lock and dam structures, the dam cannot be located near the downstream lock gate bay due to the adverse effects of the dam discharge on navigation. Also, if the dam is located adjacent to the downstream lock gate bay, provisions for chamber discharge are limited to within the downstream entrance to the chamber and/or to the side of the lock opposite from the dam.

(c) Where conditions warrant, highway or railroad bridges may be located over the lock structure. In such cases, the lock walls and/or the dam piers may serve as bridge pier supports. The elevation of low-steel on a fixed-span bridge can be minimized if the bridge is located just below the downstream gates of medium- to high-lift locks. The vertical clearance for navigation at that location will be determined by the tailwater elevation rather than the upper pool elevation. If the dam piers are to be used to support a fixed-span bridge, the best overall location of the dam is likely to be just below the downstream lock gate. Barkley Lock and Dam near Grand Rivers, KY, on the Cumberland River is an example in which the dam is located at the downstream lock gate and the dam piers provide support for a railroad bridge.

(d) From the perspective of structural stability and design, it is usually advantageous to have the majority of the lock chamber in the lower pool. This is particularly true for the condition with the lock unwatered when the lock wall stability is controlled by uplift and by water pressures tending to push the lock walls toward the chamber centerline. Additionally, this placement of the lock chamber reduces the length of the lock wall that acts as a damming surface.

(2) Factors to be considered in regard to lock and dam separation. In addition to those factors discussed elsewhere herein, the following should be considered:

(a) Typically, there will be minimum separation between the lock and dam in order to minimize construction costs and to promote efficiencies through keeping the operations and maintenance activities for the lock and the dam in close proximity.

(b) In those waterways in which the suspended sediment load is expected to be large, small separations are advantageous for transport of sediment, because limiting the size of the channel cross section will make velocity of flow greater than in a larger channel with the net result of less chance of sediment deposition.

(c) Locating the dam close to the lock may induce scouring of the lock's foundation. If the lock and dam cannot be separated sufficiently to avoid this, solutions such as paving the channel adjacent to the lock with concrete or heavy derrick stone or other special solutions may be required.

(d) As stated above, flow through the dam will influence navigation approach conditions unless there is a large separation between the lock and dam or a physical barrier to divert the flow from the approach. Model studies will best predict the effects on navigation where the dam is adjacent to the lock.

d. Geotechnical design.

(1) General. The design will be based on the geotechnical conditions at the selected site. Except for the probable necessity of rock excavation, rock foundations for dams may be the most desirable. Where rock suitable for founding structures does not exist, soil or pile foundations may be required.

(2) Properties of soil and rock. In the case of earthfill dam sections and concrete dams on earth or pile foundations, consideration must be given to the stability, drainage, and compaction properties of the fill and foundation materials, and probable settlement of the foundation when loaded. Drilled caissons and excavation-type caissons are two alternatives for unusual foundation material situations. Foundation settlement will be of major importance where two dissimilar structures such as earth-dike and masonry sections abut. Seepage from upper to lower pool must also be considered.

(3) Rock foundation requirements and other geotechnical considerations. A sound rock foundation at a reasonable depth is frequently desirable and/or available for a concrete navigation dam. Extensive geotechnical exploration and examination must be made of rock materials to determine suitable founding levels, shear strengths, and allowable bearing pressures. Foundation investigations should also include possible grouting requirements for seamy or cavernous rock, and the possible effect of saturation or passage of water through granular deposits, seams, and other susceptible material. Final verification of founding elevations for rock foundations is usually done as part of the construction contract.

(4) Geotechnical exploration of the waterway. Sufficient information must be gathered along the length of navigable waterway to determine dredging or channel realignment requirements. The characteristics of the material (rock or soil) must be identified, as this may have a significant impact on project cost.

e. Structure types.

(1) Low- to medium-head dams. The majority of the dam structure itself will likely be composed of reinforced concrete and may be founded on earth, rock, or piling--depending on the geology of the selected site. Portions not exposed to turbulent flow may be composed of roller-compacted concrete, soil cement, and earth and rockfill embankments. In many cases, overflow sections may be composed of cellular structures (sheet piling filled with granular materials, grouted riprap, etc.). The life of these cellular structures will likely be controlled by the longevity of the sheet piling, which can exceed 50 years under favorable conditions. The gates will typically be of structural steel. However, some experimentation is being conducted to use fiberglass, plastics, and similar materials for the wicket gates. The service bridge will normally be of precast, prestressed girders topped with cast-in-place concrete.

(2) High-head dams. Stability requirements for high-head dams will usually exclude or minimize use of such features as earth and rockfill embankments, soil

cement construction, and cellular structures as used on low- to medium-head dams. Typically, unreinforced gravity concrete will be used, and it is likely the dam will be founded on rock due to the typical geological and topographical conditions which lead to use of high-head dams.

f. Materials availability. The types, suitability, sources, and costs of principal construction materials should be identified in early planning stages. Of particular importance is the identification of materials that are not readily available or available from a number of domestic sources.

g. Structural criteria. Early in the design process, it is essential to develop reliable project feature dimensions and identify the operational requirements (e.g., pool elevations, ranges of pool stages, probable structure types, and foundation requirements). As the project development evolves, this information must be kept up-to-date so as to use available design resources efficiently. Divisions and HQUSACE should be involved early in the planning process to obtain the best overall design with the fewest revisions during the planning and design period.

h. Topographical considerations and mapping requirements. Accurate survey information is necessary for the development and layout of a navigation project. Information at the dam site and within the limits of the navigable reach of water will normally be required. This may require aerial photography and onsite surveys.

i. Real estate requirements and considerations. Numerous real estate considerations are associated with a navigation facility as stated elsewhere in this chapter. Project details should be developed with sufficient accuracy so as not to have to expand the number of right-of-way procurements as the project design develops.

2-12. Cofferdams and Other Temporary Construction Requirements

a. General. In addition to design and construction activities associated with permanent features, the following are some of the temporary features which must be considered in planning and design.

b. Diversion alternatives. Depending on the circumstances, the dam may be built within the confines of the streambed within a cofferdam, or it may be built within a new channel that cuts off a bendway within the existing waterway. The first method involves diversion of flow within a zone contiguous with the construction site. In the latter method, the construction site is isolated from the existing waterway until the project is completed and the flow is diverted through the cutoff channel. More detailed information relating to diversion is covered in paragraph 2-10 of this manual and in EM 1110-2-2602, which considers related diversion requirements for navigation locks.

c. Cofferdams. As mentioned above, hydraulic model studies may be needed to configure the cofferdam layout if construction is to be within the confines of the existing waterway. The cofferdam arrangement, used for construction of Melvin Price Locks and Dam and shown in Plate 14, is an example of cofferdam usage in staged construction. However, if the dam is to be constructed within a cutoff, the cofferdam is likely to consist of an earthfill embankment. An important consideration is the effective height of the cofferdam. The effective height relates to the risks that are to be taken with regard to the waterway stage at which the cofferdam will overtop and the costs of overtopping; i.e., it must be determined when the cost resulting from overtopping would be less than the cost of raising the height of the cofferdam to minimize the risk of overtopping. A more in-depth discussion of cofferdams for locks is provided in EM 1110-2-2602. Other specific guidance may be found in ER 1110-2-8152, EM 1110-2-1605, and EM 1110-2-2503.

d. Alternate methods of construction. The use of alternate methods to construct a navigation dam (other than within conventional cofferdams) may have significant advantages over conventional types of construction, in both initial construction costs and required construction time.

(1) Alternative ways to construct a dam can include construction "in-the-wet" or a reusable type of cofferdam or a combination of methods. Construction in-the-wet usually involves underwater excavation and foundation preparation (including piles). The structure is then floated into place and sunk or hoisted onto the foundation, usually in segments to maintain a manageable size. The segments may be filled with tremie concrete, or steel shells may be used which are later filled with tremie concrete. Consideration must be given to the requirements for constructing the segments in a yard and transporting them to the site, or providing a dry dock type of facility (usually near the site). Large precast piers have also been set in place with specialized equipment (Dutch tidal barrier).

(2) The dam or portions of it may also be constructed within dewatered boxes, which can be reused, or

a more sophisticated mobile cofferdam may be used which consists of a double-walled steel box that can be floated, advanced, sunk, and dewatered with a self-contained system and can also incorporate mechanized concrete forming and delivery systems.

(3) Currently (1993), advanced studies are being completed considering two alternative methods of constructing the Olmsted Dam to be built on the Ohio River, in addition to a conventional cellular cofferdam type of construction. One method would involve preparing the foundation and drive piles in-the-wet and using a mobile cofferdam to construct the dam sill and install gates and machinery in conjunction with setting precast concrete stilling basin shell elements filled with tremie concrete. The mobile cofferdam would be floated, moved into position, sunk, layered with tremie concrete, and dewatered when the tremie attained sufficient strength. Construction of that segment of the sill would be completed in-the-dry. The mobile cofferdam would then be flooded and floated and the cycle repeated. The second alternate method would be similar except large precast sill elements containing gates and machinery would be set in place on bearing beams (and later grouted) with a large special-built crane. Additionally, the fixed-weir segment of the dam would be constructed in-the-wet utilizing cellular sheet pile structures as has been common for several years.

(4) New technology or technology borrowed from other fields should be considered when determining the best way to build a dam in a riverine environment. Alternative methods may also be advantageous environmentally and hydraulically, and may minimize navigational difficulties during construction. The method used to construct a dam, the materials used, and the design chosen are closely related and must be considered together.

e. Rights-of-way. Rights-of-way remote from the project may be required for access to borrow sources, staging, and other purposes. This may or may not be a government responsibility. However, careful planning should be done to ensure that there are adequate rights-of-way at the project site so that the contractor can use standard construction procedures if at all practicable.

2-13. Environmental and Aesthetic Considerations

Environmental requirements were briefly addressed in paragraph 2-7. Unless the dam is remote from the lock, aesthetic considerations should be consistent for both the lock and the dam. Guidance relative to these subjects is essentially the same as for a navigation lock and is available in EM 1110-2-2602.

2-14. Real Estate Considerations

Numerous real estate considerations are associated with a navigation facility, and those concerning the dam site itself may form only a small part of the picture. In the investigation phases, the government may need temporary access to private property to perform surveys and foundation exploration; assess possible requirements for highway, railroad, and utility relocations; determine access-road alternatives; and for other reasons. In the site selection stage, temporary access will be needed at a number of locations to obtain adequate data for determining the best site for the structure. Project construction and/or operation purposes will require real estate for staging construction activities and for project-induced flooding of lands adjacent to the upstream channel, channel work, navigation structure, access roads, and support facilities. Surveys should be performed to identify the need to mitigate damages from levee underseepage due to changed pool conditions. Mitigation may involve compensating a landowner for estimated damages for changed industrial and agricultural land use over the project life. An alternative to mitigation may be the need for levees, pumping stations, and drainage structures to handle increased water levels and induced underseepage from changed pool conditions. Other considerations which may pose major concerns include the following:

a. Determining the types of rights-of-way required (including easements and fee title properties).

b. Establishing the entity responsible for obtaining real estate and performing relocations.

c. Estimating the lead times required to obtain rights-of-way and perform relocations.

d. Identifying lands for mitigation of changed environmental conditions.

2-15. Site Selection

Site selection is one of the most important considerations and is closely related to the other technical and procedural considerations presented above. Selection of the dam site is closely tied to selection of the site for the companion lock, and the items that are important for one are important for the other. Briefly, the selection process should consider the following:

a. A site which accommodates good approach conditions to the lock.

b. The characteristics and history of the existing stream including, but not limited to, hydraulic and hydrologic considerations.

c. The stability of the stream bed, i.e., whether the stream carries a large sediment bed load and whether the stream bed is stable or meanders.

d. Existing topographic and geologic conditions.

e. Existing uses of the waterway which may be impacted by the raised pool level, such as levees, municipal water intakes, etc.

f. The effects of the waterway on the natural environment, i.e., wildlife, vegetation, fisheries, etc.

g. Whether there is potential for hydropower development.

h. Whether other desired multiple purpose waterway usage is accommodated.

i. Whether construction at a site would produce fewer adverse impacts (environmental, flooding, etc.) than at another.

j. A site that is conducive to economical construction and operation while satisfying the above objectives.

k. A site that will provide net project benefits and is consistent with the national economic development (NED) plan, as appropriate.

Chapter 3
Access and Support Facilities

3-1. General

Requirements for access and support facilities for navigation dams are briefly described below and are closely allied to those for locks presented in EM 1110-2-2602. EM 1110-2-2602 and EM 385-1-1 should be used as principal resources in planning the access and support facilities for navigation dams.

3-2. Access to Dam Site

The principal vehicular access to the lock and dam will usually be on the lock side of the waterway because most of the operation and maintenance activities will take place there. Paved access to the end of the dam on the opposite side of the waterway is also desirable, particularly when it is necessary to deliver and receive heavy parts and equipment by truck at that location. Some means of vehicular access at that location is needed for routine maintenance.

3-3. Pedestrian Access

It is desirable to have access across the full length of the dam. Such access, however, may not be feasible if an overflow section without piers is included in the dam. In this situation, there is usually a more economical and acceptable means to access other portions of the dam or the lock from the shore than by a bridge, which would require inclusion of piers solely for its support. However, a service bridge between piers of a tainter-gated dam is essential for routine operation and maintenance activities, accessing tainter gate and bulkhead hoisting equipment for repair, and manual control of the gate operating equipment. These bridges also provide support for the electrical and other utilities required to run the hoisting equipment and may serve as a roadway for the bulkhead crane.

3-4. Elevators, Stairways, and Ladders

a. Elevators. In spite of the fact that the vertical distances between the lock wall and the service bridge on the dam and between various levels on the dam piers are typically quite large, elevators have not been used extensively. However, elevators may be used when justified because they will provide access for the handicapped, a means to lift and lower light freight, and routine access.

Further, access for the handicapped is a sensitive issue, and compliance with legal requirements for accessing certain areas may necessitate use of elevators. Current access requirements involving the handicapped are referenced in an HQUSACE memorandum, "Uniform Federal Accessibility Standards (UFAS)," dated 3 November 1986. A redetermination of "current" access requirements should always be made in planning for new navigation dam construction. For example, the Melvin Price Locks and Dam uses elevators to provide vertical lift from the lock walls (also the nongated section of the dam) to the level of the service bridge on the dam and from the service bridge to the control house located adjacent to the main lock. Similar provisions for elevators are included on Smithland Lock and Dam and will be included on the Olmsted Lock and Dam, both on the Ohio River.

b. Other vertical access. Stairways and/or ladders must be provided for vertical access where elevators are not to be used and where use of small ramps is not feasible. Stairways should be provided in lieu of ladders wherever practical when they will provide more convenient and safer access for personnel (i.e., stairways will not routinely be considered impractical simply because provision of ladder access is more economical).

3-5. Equipment Access

In some cases, the service bridge over the gated section of the dam provides the roadway or rail support for the bulkhead crane. Obviously, the bridge design would be controlled by the bulkhead crane loadings if one is to be provided but, if not, the bridge design will be based on loadings from pedestrians, tools, and support of disassembled parts of the gate hoisting equipment. In many cases, equipment will be transported to and from the dam on work flats and lifted to the level of the service bridge by a jib crane mounted on one of the dam piers. Equipment access is discussed in more depth in subsequent portions of this manual.

3-6. Operations and Maintenance Buildings

Normally, these buildings will serve jointly for operation and maintenance activities for both the lock and dam. They may be provided either at the lock and dam site or at a centralized remote location, depending on operational requirements. Generally, the purpose of these buildings will be to provide offices, shops, and storage to support the routine onsite operations and maintenance activities.

3-7. Control Houses and Operating Platforms

Although provisions are included for remote operation of gates at most modern navigation dams and should be included for future dams, local control of gates must also be provided to facilitate operation when remote operation is not practical or desired. More detailed coverage of operational and design requirements for control houses and operating platforms is provided in subsequent chapters.

3-8. Control Rooms for Remote Lock and Dam Operation

Depending on the size and layout of the project and types of structural features involved (e.g., multiple locks), it may be appropriate to operate the lock and dam gates, lock filling and emptying valves, etc., from a centralized control room having a reasonably good view of all operating features. This control room could be located on top of a dam pier which extends above the tainter gate hoisting machinery room (as at Melvin Price Locks and Dam). However, in most cases remote operation will be from a control room on the lock wall. Remote operation will require the visual aid of television cameras.

3-9. Storage Facilities for Protection of Equipment

a. Items to be incorporated in the construction. In circumstances where the government is to furnish items to the construction contractor, the government may provide temporary storage facilities when warranted. For example, used sheet piling to be incorporated in a subsequent construction phase may require storage to ensure that the pilings are maintained in suitable condition for reuse in a future construction phase. Normally, construction activities should be scheduled so that necessary storage of items to be reused in future construction phases is provided by the contractor through provisions in the ongoing contract.

b. Storage of operations and maintenance equipment and spare parts. The aforementioned buildings will usually provide for storage of operations and maintenance equipment, spare parts, etc., of relatively small size and weight. However, large items such as bulkheads and bulkhead handling equipment will be stored either on the dam structure or at some convenient location off the structure.

c. Bulkhead storage. If a bulkhead crane is provided, some or all of the bulkheads will be stored on the dam structure. If the bulkheads are to be stored off the structure, a means of accessing the dam with the bulkheads must be determined, providing for such possibilities as loss of the upper and/or lower pools, sunken barges in the approach to the dam, and other conditions which might obstruct barge access for bulkhead installation.

Chapter 4
Nonnavigation Considerations

4-1. General

a. Disturbances. Dams should be located to minimize disturbances to existing public and private installations such as roads, bridges, water intakes and other utilities, the environmental setting, and pipelines. The dam design may incorporate public recreation facilities and should also take into account the site's potential for hydropower, in case hydropower facilities need to be installed with the project or are a possibility for future installation.

b. Water-surface elevation. Changes in water-surface elevations and flow regimen of a stream due to a navigation dam may cause property damage or interference with some stream uses. However, the value of the stream for certain other uses may be increased. A navigation improvement project should provide the maximum net benefit to all interests concerned. The most cost-effective plan for the sole purpose of navigation may require modification to accommodate the critical needs of other interests. Benefits to other interests may justify a more comprehensive improvement which costs more than a single-purpose improvement. A number of important nonnavigation requirements and their effect on design and operation of a navigation improvement are discussed in the following paragraphs.

4-2. Effect on Floods

Unless the project has multiple purposes, a navigation dam usually will be constructed to the minimum height required to provide the prescribed project depth over obstructive sections of the river bed or may be supplemented by dredging to provide required navigable depth. At small discharges the pool will be nearly flat at an elevation equal to or somewhat above the natural low-water stage at the head of the pool. With larger discharges, velocity and water-surface slopes will increase and stages at the head of the pool will rise if the lower end of the pool remains fixed at the height of the dam. If the dam has a height considerably less than the stage of maximum floods, it may be so deeply submerged at high stages as to have no appreciable effect upon the larger floods. If the height of the dam approaches or is greater than the maximum flood stage, lands not previously subject to flooding may be damaged by large flood discharges. Damage from flooding may be minimized by setting the pool at the dam so that the sloping water surface profile will provide only the desired project depth over the controlling obstructive section. This elevation will be determined by the slope which would produce the limiting velocity for navigation in the lower portion of the pool.

4-3. Effect on Drainage

a. Discharges. Throughout the length of a navigation pool, the water surface will be held permanently above the natural low-water stage. In some cases, stages at the dam may be permanently above the highest natural flood stages. At the head of the pool, stages will fluctuate between normal pool level and flood stages in substantially the same manner as normal open-river stages. The sustained increase in stage above natural low water may interfere with the discharge of sewers, culverts, and tributary streams which formerly discharged freely at low stages. Deposits of sludge or silt due to reduced velocities may block sewers and culverts and raise tributary stream bottoms to the point where flood heights are affected. In the case of sewers and drainage outlets from drained areas or areas protected by levees, pumping may be required to provide satisfactory drainage. In cases of permanently submerged gates on local flood protection projects, it may be difficult to inspect these locations adequately during floods for debris that would block the gate from closing when needed. The use of a flap gate at the discharge end of the culvert and a trash rack at the upstream end plus periodic inspection of the gate area during low discharge periods can help alleviate this problem. Where pumping plants already exist, the pool stages may require increased use of pumps and pumpage against increased average head. The sustained increase in stage may also interfere with underground flow of surplus water from agricultural bottom lands into the stream. Additional ditching and pumping may be required to maintain satisfactory agricultural drainage.

b. Legal considerations. As the effects of navigation pools upon drainage do not involve direct invasion or overflow of lands, they have been defined by court decisions as "consequential damages," which are not compensable in condemnation proceedings. However, damages of consequential nature have been reimbursed by special acts of Congress in several instances. In view of the precedents established by such legislation, probable damage to sewers and drainage should be evaluated as a cost of the project, and should be held to a practicable minimum. Damage can be averted or minimized by selecting dam locations upstream rather than downstream from important drainage outlets and tributary streams.

4-4. Water Quality

Aeration provided by turbulent flow through dams can aid in maintaining the dissolved oxygen requirements for environmental needs. Low flow requirements should also be met.

4-5. Water Supply

Where water supplies are drawn from the stream, a navigation dam will be of value in providing adequate depth at the intake.

4-6. Mosquito Control

In some latitudes, whenever a relatively stable pool is created, a suitable environment is provided for mosquito breeding, particularly if floating debris, dead brush, or aquatic vegetation is allowed to accumulate in shallow marginal areas. In localities where mosquito breeding exists, consideration should be given in design of structures to the desirability of fluctuating pool levels in order to keep pools free of drift and undesirable vegetation and to strand mosquito eggs, larvae, and pupae associated with marginal vegetation and flotage. In most cases, projects can be designed for such operation without significant disadvantage to the primary function of the project.

4-7. Environmental

The National Environmental Policy Act (NEPA) of 1969 (PL 91-190) established a broad national policy directing federal agencies to maintain and preserve environmental quality.

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

b. Research. The following research efforts were initiated by the Corps of Engineers in order to gain more knowledge about and better comply with the preceding environmental legislation.

(1) Dredged Material Research Program (DMRP). The DMRP was completed by WES in 1978. The program's objective was to determine the environmental effects of dredged material disposal and to develop methods for eliminating or minimizing any adverse effects.

(2) Dredging Operations Technical Support (DOTS). The DOTS program was established in 1978 at the conclusion of the DMRP to assist all Corps elements in the implementation of DMRP results. The program maintains WES's capability of responding to requests for assistance from the Corps elements on all environmental problems associated with dredging, dredged material disposal, and habitat creation.

(3) Environmental and Water Quality Operational Studies (EWQOS). The principal objective of EWQOS, initiated in 1977, is to provide new or improved technology for planning, design, construction, and operation of Corps civil works projects to meet environmental quality objectives in a manner compatible with authorized project purposes.

c. Environmental problems. Problems that must be considered during navigation dam project development are excessive sedimentation; resuspension of contaminants; increased water temperature; water table effects; excavated material; impacts on aquatic, wetland, and territorial habitats; interruption of migratory routes; modification of riparian habitats; disruption of breeding or nursery areas; increased turbidity; impacts upon wetlands; changes associated with the formation of bendway cutoffs; and any necessary mitigation of damages.

d. Reference. EM 1110-2-1611 contains in-depth coverage of environmental considerations for navigation dam projects.

e. Fish and wildlife.

(1) Conditions for propagation and survival of fish and wildlife may be altered extensively by a navigation dam. Permanently raised water levels invariably destroy spawning areas, nesting grounds, and dens. Clearing of

brush and timber further reduces wildlife habitat, and may leave inadequate cover and feeding grounds for survival of existing wildlife. However, in many cases, equally suitable spawning, nesting, denning, and feeding areas may be formed at higher elevations, and habitat for some species actually may be improved or increased. Some types of dams may be barriers to the movement of migratory fish, in which case it may be necessary to provide fishladders or other facilities. In other cases, the dams may serve as barriers to rough fish, to the general benefit of more desirable species. In general, stabilized pool levels will greatly reduce the danger of fish being stranded by low water during the navigation season. Elimination of fluctuations in the zone between normal pool level and natural low-water level will also reduce the hazard to wildlife of having nests and dens flooded out during the breeding season. Large stabilized pools may also provide waterfowl resting or feeding areas suitable for management as public hunting grounds and refuges. Under proper control, other significant benefits may be realized from the natural resources preserved or provided by the pools.

(2) In designing structures, consideration should be given to the recommendation of conservation interests as to the effects of various pool levels, dam locations, and operating procedures on valuable wildlife resources. For example, location of a dam above or below a tributary stream may have a decided bearing on the wildlife values of the tributary basin. In some cases it may be practicable to modify a project, as designed for its primary purpose, in order to minimize possible losses to fish and wildlife or to facilitate a method of operation which will better serve fish and wildlife interests. For example, in climates where ice will prevent navigation for several months per year, the best method of operation for navigation alone might be to drain the pool in order to reduce the volume of ice to be passed during the spring breakup. However, the benefits of retaining a high pool level to prevent crowding of fish under ice cover, preserve access to dens and shelters of aquatic fur-bearers, and protect aquatic vegetation used by fish and wildlife for food and cover might justify design of the project on the basis of holding full pool or near-full pool and providing for passage of the larger volume of ice. Operational drawdowns at high discharges during the navigation season are also objectionable to fish and wildlife interests, but are less apt to be harmful to fish and wildlife than winter drawdowns.

4-8. Recreation

Although the effect of a project on existing recreational facilities and natural recreational areas is usually an important consideration, there may be times when the cost of purchasing or replacing such facilities must also be considered. Attention should also be given in the planning stage to project visitation and to the possibility of converting access roads, buildings, and other facilities used in project construction to recreational use upon completion of the project, in recognition of the fact that the impoundment of a large body of water often improves the area's recreational potential or creates new opportunities for recreational development. Particular attention should be given to project features, such as beaches and boat facilities, which can be developed most economically before the pool is filled.

4-9. Hydropower

a. General. In the case of comparatively high-head dams where the upper pool is above maximum tailwater, the possibility of power development is evident. In the case of low-head dams where a usable power head is available for extended periods, the value of seasonal power to meet coincident seasonal power demand may warrant consideration of a power installation. In evaluating power possibilities, leakage through the lock and dam and water required for operation of the lock must be subtracted from the stream flow. The most suitable type of power development at a navigation dam usually will be a run-of-river plant, possibly with a limited drawdown for daily or weekly pondage operation. Peak power operation with large and rapid daily fluctuations in discharge can be objectionable to navigation. To reduce the effect of powerhouse operation on navigation activities, the power facilities should be on the side of the stream opposite the lock or locks when practicable. The power value of a high dam, with seasonal storage to increase low flows, may warrant consideration of a large power drawdown with a supplemental lock and dam in the upper end of the pool to maintain navigation during drawdown periods. The best plans for development of power or navigation alone may be in conflict, and a dual- or multiple-purpose development will involve a compromise imposing some degree of restriction on all uses of a stream. Plans for operation must be sufficiently flexible to meet seasonal variations in weather and stream flow and to permit development of maximum overall benefits.

b. Minimum provisions for hydropower. Current regulatory requirements which control whether hydropower is included in a project are outlined in ER 1105-2-100. Hydropower should be included when it contributes to the NED plan. In determining the NED plan, studies should consider the economic benefits resulting from project operation. In the past, some projects have

included minimum provisions to more readily permit installation of hydropower in the future should it become economically justified.

4-10. Zebra Mussels

a. Description. The zebra mussel (*Dreissena polymorpha*), a native of the Black and Caspian Seas of Central Asia, matures to a length of only 2 in. However, it reproduces in great numbers; in fact, one female can release 30,000 to 100,000 eggs per year. Zebra mussel larva (veligers) are free-swimming for about 2 weeks before settling on hard surfaces where they mature. Once settled, they become firmly attached and are difficult to remove. This ability to firmly attach to surfaces in large numbers makes this mussel a major liability to users of the waterways. In addition, the mussel has few natural enemies, and it thrives in freshwater areas (especially in flowing water) where there is a plentiful supply of food and dissolved oxygen.

b. Introduction into North American waters. From Asia, the zebra mussel spread to European freshwater ports and then to most of Europe's inland waterway system. Now (1993) the mussel has spread through most of the major rivers and lakes in the north-central and northeastern United States. Although the exact mode of introduction is not known, it is suspected that a vessel originating in an overseas freshwater port took on ballast water which contained juvenile zebra mussels or larva. Upon entering the freshwater ports of the Great Lakes, the vessel discharged its ballast water, and the mussels then had a new territory to colonize. It appears that this mussel will eventually spread to all the major inland U.S. waterways.

c. Problems created.

(1) In the short time the zebra mussel has been in the United States, it has fouled water intake structures for water treatment plants, power plants, and industrial water systems; crushed historic sunken vessels; made beaches almost unusable; damaged boat hulls; and deprived fish of a normal food supply. These mussels are also invading navigation lock and dam structures in ever-increasing numbers. Thus, as the mussel population increases and spreads, the often serious and costly problems which its presence causes will only worsen. The severity of the problem is emphasized by the fact that several Great Lakes power plants spend more than $250,000 annually

on zebra mussel control. For example, Detroit Edison removes 140 tons of mussels a year from one power plant on Lake Erie. In another example, a water treatment plant serving a city of 50,000 people was forced to suspend service because its main intake line was clogged with mussels. In the next 10 years, the cost of fighting and controlling the mussel could reach several billion dollars.

(2) Zebra mussel attachment in sufficient numbers and in a particular location could cause serious operational problems for and malfunction of any one or more of the following navigation-dam components: spillway gates, including side seal rubbing plates and sill plate; spillway gate slots and spillway bulkhead slots; stilling basin relief holes; and navigable pass or submergible gates and recesses. The problems include blockage of relief holes and vents, blockage of water flows through trash racks, and interference with spillway gate sealing and seating. Spillway gates and other steel surfaces are especially vulnerable because the mussels create an accelerating corrosive environment.

d. Control methods. The following procedures have proven to be effective in removing the zebra mussel from concrete and steel structures: hand or power scraping, high-pressure water jetting, suction pump vacuuming, thermal shock (elevating water temperature to over 90 ºF), drying with hot air, and use of biocides. Antifoulant coatings, both toxic and nontoxic, have been used on structures such as intake trash racks. The coatings will need to be renewed in about 5 years. Steel structures such as trash racks should be designed to be removable for easy recoating and/or mussel removal.

e. Corps of Engineers actions. WES is engaged in the Zebra Mussel Control Research Program, a multimillion dollar, Congressionally mandated project to investigate the environmentally sound control of zebra mussels in and around public facilities. The program will monitor the spread of the mussels, test certain antifouling coatings in infested areas, and explore the use of new control devices, methods, and designs. The gravity of the zebra mussel problem, as it relates to the operation, maintenance, and longevity of Corps navigation lock and dam structures, makes it a "must" that all future designs take into account the necessity for mussel control and elimination. WES is the best source of information at the present time (1993).

Chapter 5
Types of Navigation Dam Structures

5-1. General

A navigation dam is composed of one or more types of structures that operate together to dam a pool of water. The components will be dictated by site flow conditions, geotechnical considerations, operational and maintenance requirements, construction considerations, and requirements of the user (the towing industry). Spillway types normally provided for navigation dams include the following: gated--nonnavigable, gated--navigable, fixed crest--overflow, and fixed crest--nonoverflow.

5-2. Gated Nonnavigable Spillway

The type of gate selected also controls the dam sill and associated piers. Gate types typically used for a non-navigable spillway include tainter (radial) gates, hinged-crest gates (Bascule, Pelican, and flap), vertical-lift gates, roller gates, and wicket gates. (Gate types which have been used in the past but are not recommended for use, except in special situations, are bear trap gates, drum gates, and inflatable rubber gates.) The following discussion describes these gate types and presents advantages and disadvantages of each.

a. Tainter gates (radial gates).

(1) General. The radial gate most commonly used on navigational projects is the tainter gate (see Figure 5-1 and Plates 2, 6, 8, and 13). In its simplest form, a tainter gate is a segment of a cylinder mounted on radial arms that rotate on trunnions anchored to the piers. Because of its simple design, relatively light weight, and low hoist-capacity requirements, the tainter gate is considered one of the most economical and most suitable gates for controlled spillways. The use of side seals eliminates the need for gate slots that are conducive to local low-pressure areas and possible cavitation. Currently, the preferred practice is to carry the water load with a skin that transfers the load to vertical structural sections. The load is then transferred to deep horizontal beams (usually three) which then transfer the load to the trunnion arms, the trunnion yoke and hub, and the pier trunnion girder and anchorage. Several navigational projects (for example, Cannelton and Markland) use a "stressed skin" tainter gate, and although this gate may be somewhat lighter, it is more difficult to design and construct. The tainter gate is raised and lowered by wire rope (chains are also used at
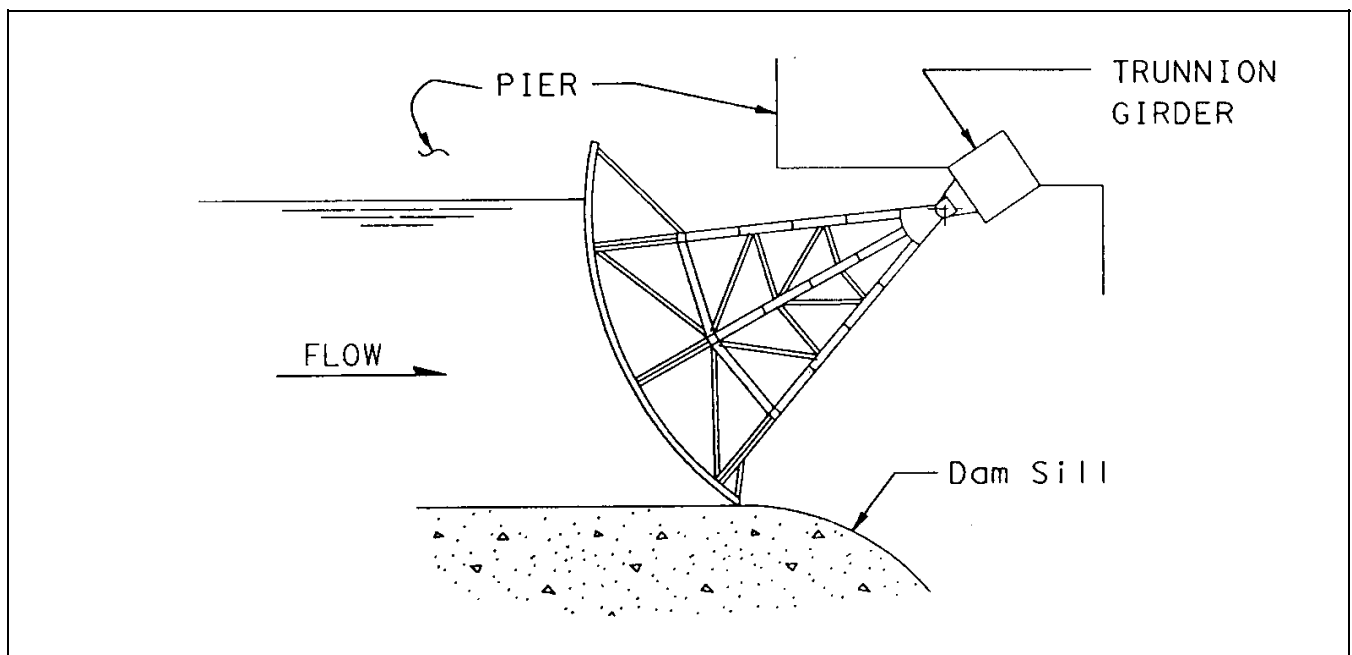
Figure 5-1. Tainter gate

older installations) attached at both ends to avoid introducing torsional stress into the gate. Gates are usually manipulated by individual hoists, one at each end of the gate. Counterweights on smaller gates will reduce required power but will add to the total weight of the structure. Tainter gates built to heights of 75 ft and lengths of 110 ft have been used for navigation dams.

(2) Gate and spillway geometry. In many cases, it may be advantageous to use same-width spillway bays and lock chamber so that the same emergency closure may be utilized on both. 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. Trunnion elevation is set above most floods. Typical trunnion submergence allowed for trunnion girders is a maximum of 5 to 10 percent of the time. When in the closed position, the gates should have at least 1 ft of freeboard above the normal upstream pool. On large pools where fetch for wave setup is large and water conservation is important, more than 1 ft may be required. Gates should be designed to clear the highest flood with allowance for floating debris. Typical clearance is 1 to 5 ft 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 dispersion of flows through the protected areas. For spillways in such locations, the maximum gate-opening height would be set at 1 ft 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 (although speed of operation usually precludes such action). 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 the 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. For design guidance, refer to EM 1110-2-1603, EM 1110-2-1605, and EM 1110-2-2702.

(3) Advantages. Tainter gate installations, as opposed to other types, have the following advantages: lighter lifting weight with smaller hoist requirements; 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); less time required for overall gate operation (more than one gate can be operated at the same time); and favorable discharge characteristics.

(4) Disadvantages. Tainter gate installations, however, have the following disadvantages: radial arms requiring more pier concrete and foundation concrete, i.e. longer and higher structure; the encroachment of the radial arm on the water passage; the necessity for long radial arms where the flood level, to be cleared, is extremely high; and relatively tall, narrow piers which may not perform well during large magnitude seismic events, especially if the motion is applied perpendicular to normal river flow.

(5) Radial gates. Gates of a configuration similar to that of tainter gates, but which are raised or lowered with hydraulic cylinders instead of cables, are usually referred to as radial gates. In Europe, these gates are now normally used in lieu of cable-hoisted gates. Besides sharing the advantages listed above, the radial gates may be more economical.

(6) Reversed tainter gates. Reversed tainter gates are sometimes used (especially in Europe). This configuration transfers the water load by putting the steel trunnion arms in tension and the concrete pier in compression, which is advantageous. However, the overall length of pier and stilling basin will usually be increased. Passing pack ice and debris is not accomplished as well as with conventional tainter gates. The presence of ice, debris, and trash in U.S. waterways would probably preclude the use of the reversed tainter gate.

(7) Submergible tainter gates. Submergible tainter gates were developed to allow passage of ice without having to use large gate openings. Two types have evolved, one in which the top of the gate can be lowered below the normal upper pool elevation and the piggyback gate, in which a shaped lip on the top of the gate or a double skin plate can be used to keep the flow off the back of the gate. Hoist loads are much greater in deep submerged positions and must be considered in machinery costs. Vibration of submerged tainter gates has been so prevalent that such gates should not be considered without the concurrence of Corps of Engineers Civil Works, Engineering Division (CECW-ED).

(8) Tainter gate piers. Tainter gate piers are concrete with a precast/prestressed concrete or steel trunnion girder anchored into the pier with post-tensioned anchors. The pier thickness varies with height and loading conditions but is usually 10 to 15 ft. The gate sill is also

concrete with embedded metal sill plates. The pier and gate sill are separate gravity type structures on larger projects; however, they may be combined into one unit when maximum width-of-monolith requirements are met. These combined units may be shaped as a T, U, L, or some combination of these shapes. The overall size of the structure must conform to requirements designed to meet constructibility limitations and to control cracking. Most massive concrete structures require a special study (nonlinear, incremental structural analysis or NISA) which must be accomplished in accordance with the requirements of ETL 1110-2-365 (see Chapters 9, 11, and 12 also). Other considerations, such as batch plant size, navigation during construction, and cofferdamming concerns, may control the size of the monoliths.

b. Hinged-crest gates.

(1) General. Hinged-crest gates are known by a variety of names including Bascule, Pelican, and flap gates. These gates are hinged at the base to a dam sill and are raised to retain pool and lowered to pass flows. They can be straight or curved to fit the dam sill crest when in the lowered position. The plate is reinforced with vertical and horizontal members and is fitted with a torque tube at the base or separate hinges. These gates are normally sealed at the base and edges when in the raised position (see Figure 5-2).

(2) Automated operation. Automated operation of hinged-crest and Bascule gates may be considered. Such automation was included on the hydraulically operated Bascule gate on the Jonesville Dam, which began operation on the Ouachita-Black Rivers Navigation System around 1970. A signal initiating from a floatwell causes the hydraulic cylinders to lower the gate to compensate for flow which would cause the upper pool stage to exceed the desired level. The cylinders raise the gate when the upper pool begins to fall below the desired level. Some problems were encountered with the Jonesville Dam automated operation when there was a lot of wave action on the river, since the signaling process was extremely sensitive to constant cyclic variation in water level. (Stillwater operation did not present a problem.) Also, there was leaf vibration initially, and spoilers had to be fabricated on the gate leaf to prevent excessive oscillation.

(3) Torque tube construction. Where the gate is constructed with a torque tube, the torque tube is supported on bearings at intervals along the gate. The gate can be raised or lowered by a crank arm powered by a hydraulic cylinder. A hinged-crest gate can also be supported by a number of separate hinges, with an operating stem (a screw stem or hydraulic cylinder) attached to one or both ends of the gate at the top. As the stem is pulled, the gate rises. The screw or cylinder is supported
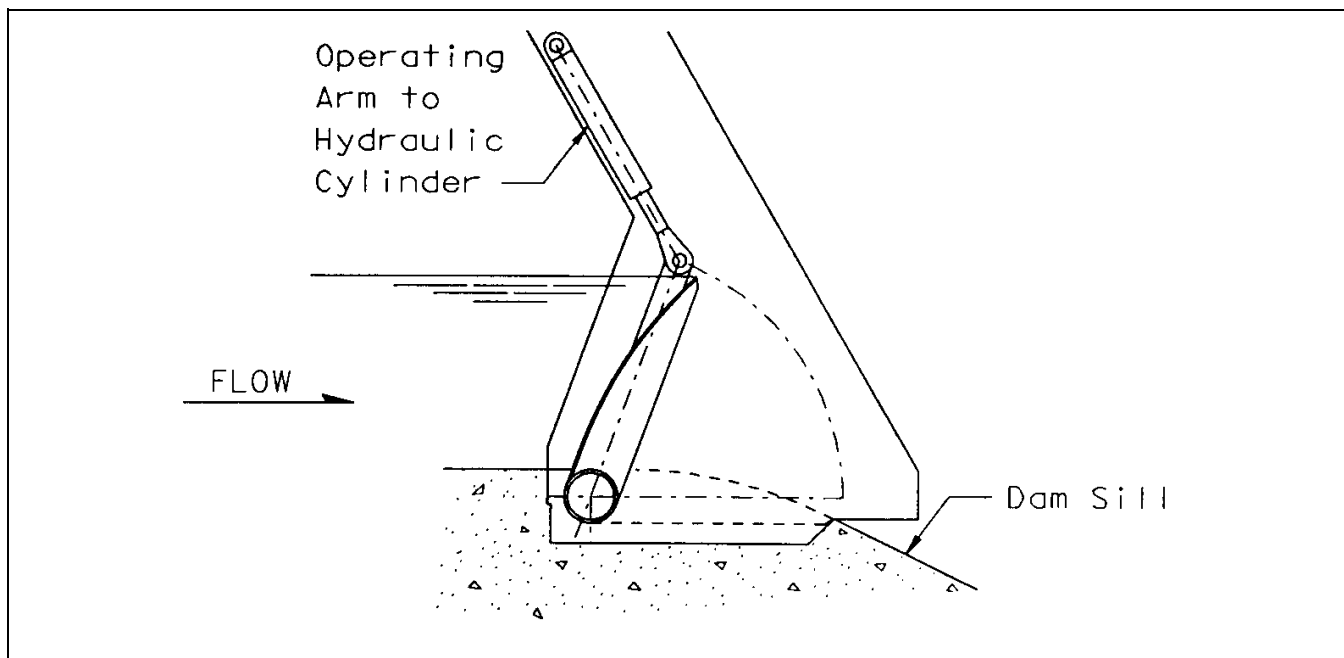
Figure 5-2. Hinged-crest gate

so that it can rotate to maintain alignment with the rotating gate. This same kind of gate can be operated by means of hydraulic cylinders mounted beneath it which push it to the up or closed position. One, two, or more cylinders can be used depending on the gate length. The hydraulic piping to the cylinders is interconnected so that the cylinders will move in unison.

(4) Gate design. The design of the crest gate itself and the means of actuation normally depend on the location of the gate, the application, the size of the gate, and the head on the gate.

(a) The simplest form of hinged-crest gate is the flat stiffened plate hinged at the bottom and operated by a screw stem or hydraulic cylinder connected to the top of the gate at one end. This type of hinged-crest gate is limited to approximately 35 ft of length by 8 ft of height. Gates that are longer or higher than this may require an actuator at each end.

(b) The torque tube-style hinged-crest gate, which uses the torque tube along the invert with the actuator mounted in a compartment in the abutment, produces an overflow between the abutments with no obstructions. The operator may be enclosed in a chamber where it is not exposed to the weather. Torque tube-style hinged-crest gates are normally limited to approximately 35 ft long by 10 ft high because of the size of the torque tube required for larger gates. However, 5-ft-high gates as long as 200 ft have been constructed with operators provided at each end.

(c) The hinged-crest gate with hydraulic cylinders underneath can be made in much longer lengths. The gate can be made in a number of sections (joined in the field) to total 200 ft or more. Hydraulic cylinders are placed at intervals beneath the gate to raise and lower it. The main advantage of this type of gate is the long lengths of gate that are possible. The disadvantage is that there must be a drop in elevation downstream of the ogee crest to be able to mount the cylinders, or the cylinders must be mounted in pits or holes downstream of the crest gate.

(d) The standard Bascule gate design consists of a torque tube with a leaf extension. The gate is rotated approximately 70 deg from fully raised (closed) to fully lowered (open) position. Bearings anchored at intervals along the length of the spillway support the torque tube. A lever arm extends from the torque tube and is positioned by a hydraulic cylinder operator. The standard Bascule gate is practical up to heights of approximately

10 ft, depending on the length of the gate, operator arrangement, and structural limitations.

(e) The Pelican gate usually proves to be more economical than the standard Bascule gate for many applications and is especially suitable for greater heights and lengths. Gates over 13 ft high have been built, and heights of over 20 ft are feasible. The Pelican gate design consists of two curved plates with internal braces and vertical bulkhead ribs forming a strong closed-shell structure. The ribs extend through the bottom of the gate and form supports for the gate hinge pins. The stationary portion of the gate bearings consists of a series of bearing supports anchored at intervals along the length of the spillway. A small diameter pipe section may be welded to the bottom portion of the gate to make contact with the longitudinal rubber seal at all gate positions. Hydraulic cylinder operators are located either on piers at the ends of the gate or on the downstream side of the gate.

(5) Dam sills and piers. Hinged-crest gate dam sills and piers are concrete. The sill and pier are normally constructed as a single monolith. Operating machinery is normally mounted on tall piers or housed in a watertight chamber so that it is not submerged, but current technology allows submerged hydraulic cylinders.

c. Vertical-lift gates.

(1) General. The vertical-lift gate, with wheels (rollers) at each end, 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 (see Figure 5-3). Reference is made to EM 1110-2-2701 and EM 1110-2-1603 for design of vertical-lift gates. 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 or cables (cables are generally desirable). Piers must be extended to a considerable height above high water in order to provide guide slots for the gate in the fully raised position. Vertical-lift gates have been designed for spans in excess of 100 ft. High vertical-lift gates may consist of two or more sections in order to facilitate storage or ease passing of 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. Historically, gantry cranes traveling on the spillway deck have been the standard method of operation for vertical-lift gates; however, fixed hoists may be justified, especially if speed is important or remote control is desired.
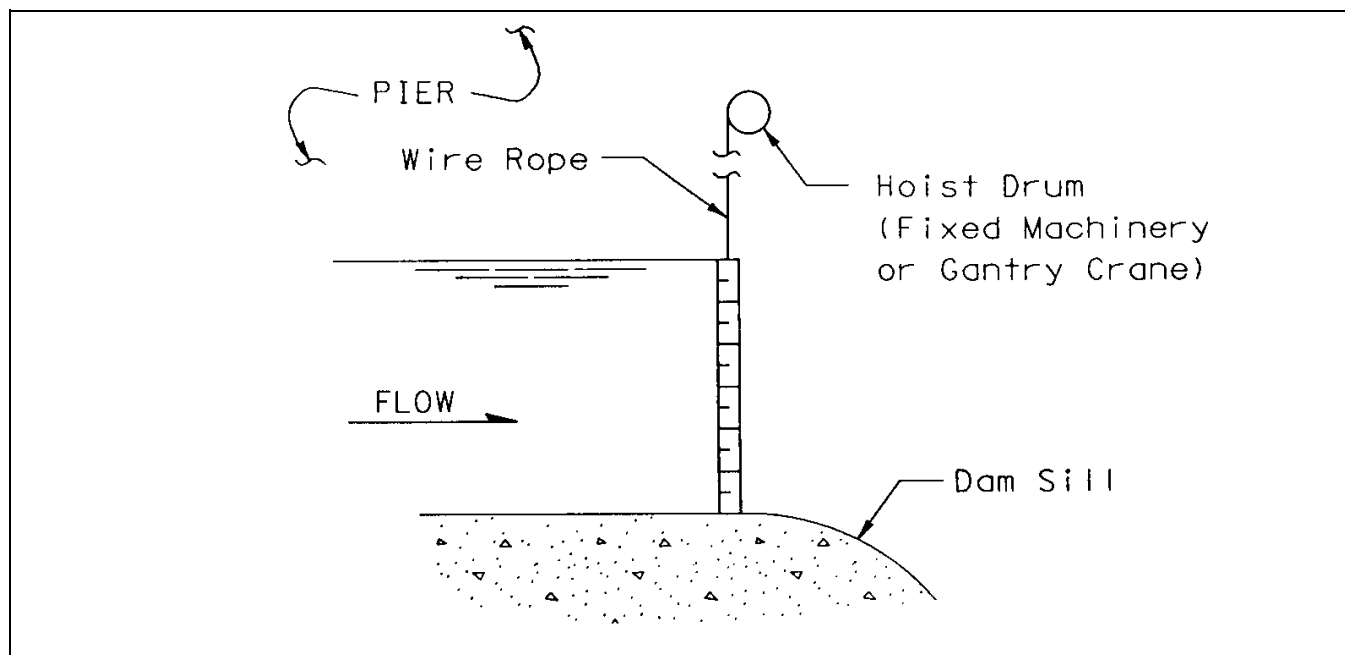
Figure 5-3. Vertical-lift gate

(2) Flow regulation. Regulation of flow is accomplished by means of single-section gates with variable discharge between the bottom of gate and the sill, and multiple-section gates consisting of two or more sections in the same slot with variable discharge between the sections or beneath the bottom section. The multiple-section gate may or may not be equipped with a latching mechanism permitting operation as a single-section gate when desired.

(3) Spillway discharge. The spillway discharge is controlled by raising the gates or the individual sections, as necessary, by increments. Dogging devices operated from the piers at the deck level engage projections spaced at intervals on the gate, permitting disengagement of the crane after the gate is raised to the proper elevation to give the required discharge.

(4) Gate types. Vertical-lift gates may be classified according to the method used to transfer the water load to the spillway piers, as follows:

(a) The fixed-wheel gate has wheels that revolve on fixed axles, which are either cantilevered from the body of the gate or supported at each end by the webs of a vertical double girder attached to the gate framing. The wheels may also be mounted by pairs in trucks which carry the wheel loads through center pins to the end girders attached to the gate frame.

(b) The tractor gate is equipped at each end with one or more endless trains of small rollers, which are mounted either directly on the end girder or on members attached to the end girder.

(c) The stoney gate has an end bearing consisting of a train of small rollers between the downstream flange of the end girder and the track on the pier. Since the rollers revolve in contact with both girder and track, the roller assembly moves in a vertical direction only half as fast as the gate and must be supported independently.

(5) Most common gate. Of the types mentioned, the fixed-wheel gate is the most common. It is adapted to long spans since provision can easily be made for rotation of the end bearings due to deflection of the gate body. It can transfer heavy, moving loads to the piers without the close track tolerances necessary for tractor or stoney gates. With cantilevered wheels, a gate slot of minimum depth can be used.

(6) Advantages. The advantages of a vertical-lift gate installation are numerous: it reduces pier dimension in upstream-downstream direction; its gate design is simple; it provides a clear gate opening with no encroachment, when raised, of any part of the gate structure on the water passage; it is more adaptable to extreme pool fluctuations because it is lifted bodily out of the water; it

eliminates design of complicated prestressing systems; and it may allow for other than in-place maintenance.

(7) Disadvantages. The disadvantages encountered in the use of vertical-lift gates include: a heavier lifting load, which requires greater hoist capacity; storage or pier-height requirements, which may necessitate use of a sectional gate; a more labor-intensive operation; greater time required for gate operation if only one crane is provided; and gate slots that can lead to cavitation and debris collection.

(8) Dam sills and piers. Vertical-lift gate dam sills and piers are concrete. The sill and piers may be constructed as separate monoliths or as a single monolith. Pier thicknesses of 8 to 15 ft are normally used, depending on gate width and height. Steel members should be embedded to serve as a sill plate and as bearing and armor members in the guide slots.

d. 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 (see Figure 5-4). 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 or cable wrapped around one end of the cylinder and operated by a hoist permanently mounted in the pier. The rolling movement of the gate and the limited amount of frictional contact at the sealing points permit comparatively fast operation with a small expenditure of power. Roller gates have been built with a damming height of 30 ft, with lengths up to 125 ft on pile foundations and 150 ft on rock foundations. Roller gates are efficient in their power requirements and can be used for wider spillway bays than other types of gates. However, complexity of construction and the maintenance required by the hoisting and roller system are disadvantages of this type of gate. Sills and piers for roller gates are comparable to those of other gates of similar height and width.

e. Wicket gates.

(1) Wicket-type gates have been utilized for navigation dams for over 100 years. These gates are now normally considered for navigable pass dam spillways, but they will also function as nonnavigable spillways. Although several types have been utilized in the past, current new projects utilize a bottom-hinged wicket gate with consideration given to chanoine-type wickets. The gates can be lifted into position with a hydraulic cylinder applying force to the downstream side or to a crank, or they can be hoisted into position with a boat or gantry-operated crane or winch. Safety considerations and ease of operation during variable river stages and climatic conditions have generally led to the requirement for hydraulic cylinder operators. These cylinders may be located in a dry gallery or in a wet recess; however, silt must be excluded from any recesses. The wickets are generally held in an up position with a prop or strut which slides in a hurter track (see Figure 5-5). This allows the cylinder piston to be retracted except during operating cycles. A gate with the cylinder rod attached directly to the back to hold the gate in position, as well as to raise and lower it, is also being tested.

(2) Wickets constructed in the past were generally of steel or iron framing with timber leafs. Steel is the material most suitable for new construction, with composites, stainless steel, or aluminum as possible alternates. Wickets which are hinged at the base have the advantage of simplicity and cannot be "flipped" up by thrust from an upbound tow and then held partially up by river currents. The chanoine-type wicket is hinged just below its center point to a collapsible horse and held in place with a prop. This type of wicket is raised with the leaf in a horizontal position and then tilted into position by the force of water. This method requires less hoisting force than other methods, but the assembly is more complex and the wicket can be "kicked" up by a tow and held partially up under extreme conditions.

(3) Wicket gates are planned for the Olmsted Locks and Dam project on the Ohio River. These gates are bottom-hinged and are raised hydraulically. They are nominally 10 ft wide by approximately 26 ft long (see Plate 4). Wider gates are feasible. Advantages of wicket gates are low initial cost of construction, lighter weight (which allows offsite maintenance), variability in controlling pool, adaptability in incorporating redundant or protective features such as multiple ways to raise the wicket (hydraulically, with boat-operated backup), and breakaway props or dogging devices to limit damage in the event of towboat impact. Wicket dams are less subject to damage in high-seismic-force areas than a dam with piers, and are also more aesthetically appealing.

(4) The main disadvantage of wicket gates is the difficulty in providing for maintenance of the wicket assemblies. Maintenance may be accomplished with an unwatering box which is placed over the sill and dewatered to allow removal and replacement of one or more wickets at each setting. Proper floating plant,

to Hoist Drum on Pier

Wire Rope
or Chain

Ring Gear &
Rack

FLOW

Dam Sill

Figure 5-4. Roller gate

Wicket Gate

FLOW

Prop

Hurter Track

Dam Sill

Wicket Gate
Operator

Hydraulic
Cylinder
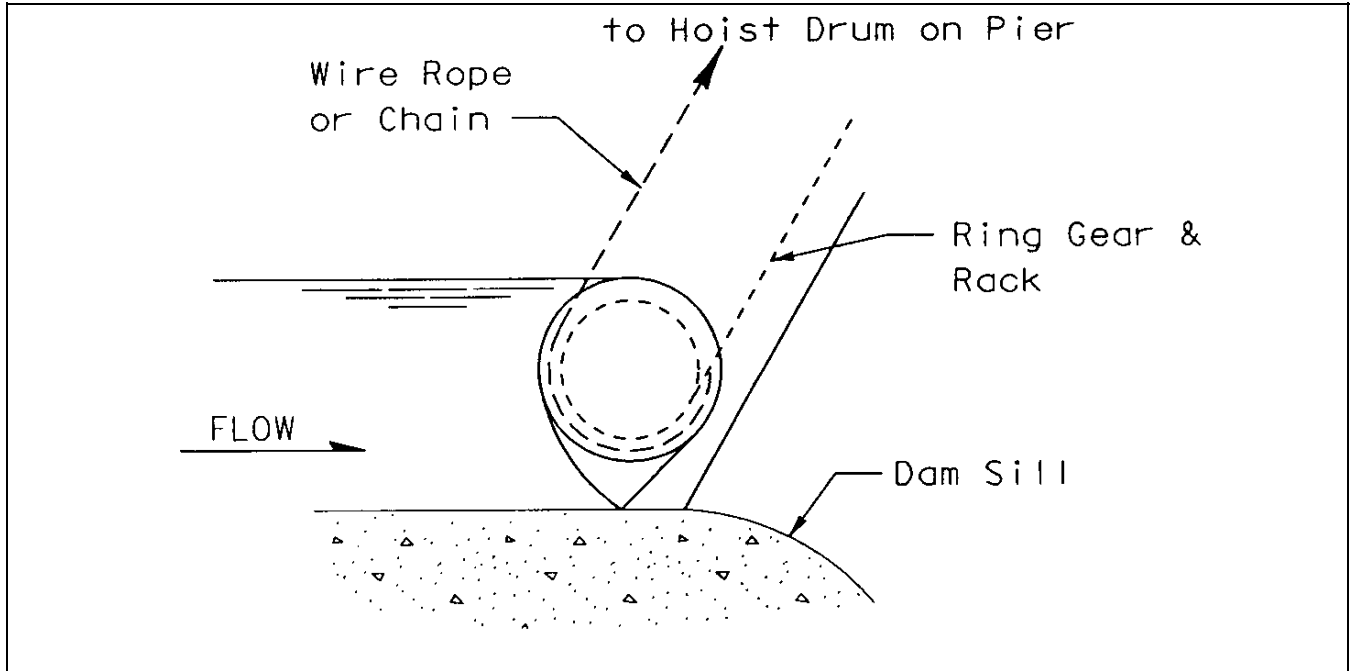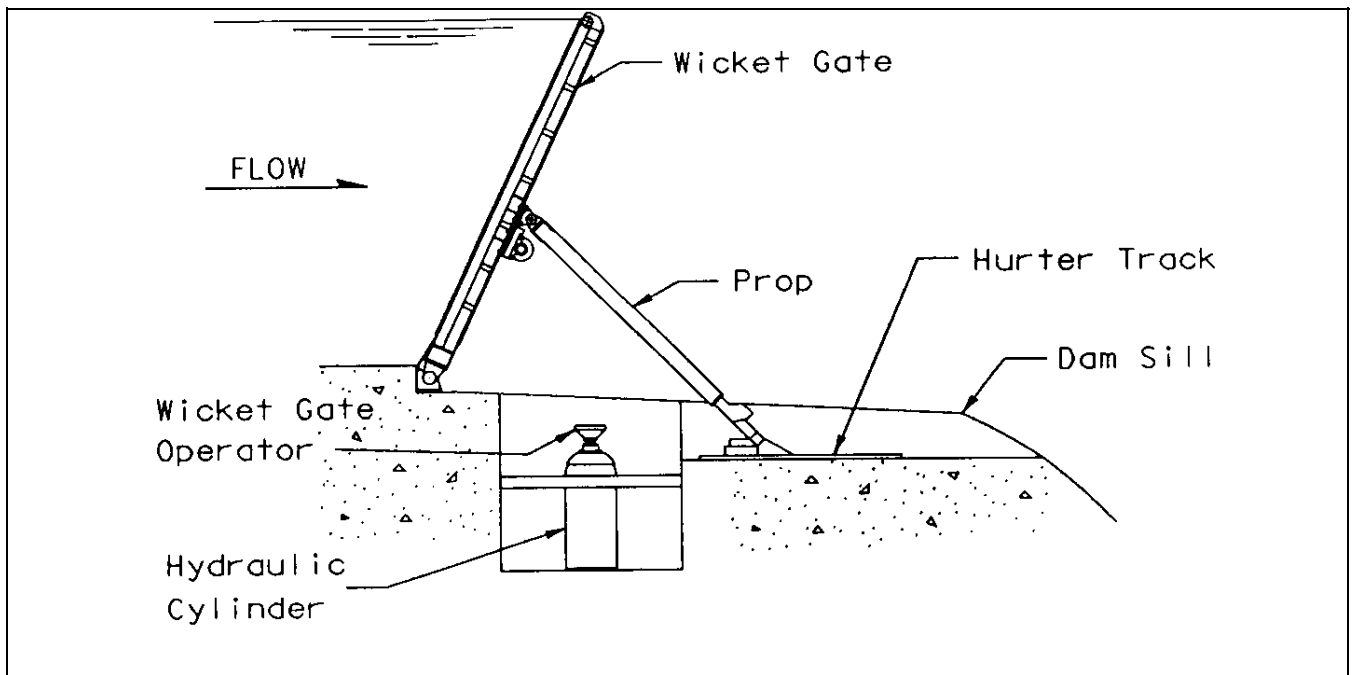
Figure 5-5. Wicket gate

anchorage, and equipment must be provided. This is most easily accomplished by providing quick change-out designs and spare wickets and operating machinery.

(5) Wicket dam sills are concrete. Piers are not required. The length of sill monoliths is controlled by cracking and constructibility requirements.

f. Bear trap gates, drum gates, and inflatable rubber gates. These gates have been utilized but are generally not recommended for current consideration. A brief discussion is included for reference (see Figures 5-6, 5-7, and 5-8).

(1) Bear trap gates consist of two leaves. When in the lowered position, the upstream leaf overlaps the downstream leaf. The gate is raised by applying upper pool pressure to a chamber under the leaves. This pressure, sometimes supplemented with air or hydraulic cylinders, raises the dam gate. These gates generally retain a pool differential of 20 ft or less and are normally about 90 ft wide. They are ingeniously conceived but can prove difficult to maintain. Silt or sand deposits in or under the gates are particularly likely and may make it impossible to fully lower or raise the gates.

(2) Drum gates are generally operated on a principle similar to the bear trap. The drum gate may be constructed as a segment of a circle and hinged on its downstream end. A watertight sill chamber is provided for the gate. To raise the gate, upper pool pressures are introduced to the chamber. This force may be supplemented by flotation chambers or hydraulic cylinders. The major difficulty encountered with this gate is the necessity to exclude silt and sand from and maintain seals on the chamber.

(3) Inflatable rubber dams are rubberized fabric tubes which are anchored to a sill and inflated to form a dam. These dams are limited to very low-head project usage, are subject to puncturing and vandalism, and are not recommended for major projects.

5-3. Gated Navigable Spillway

Navigable pass spillways permit the passage of tows over dams without the locking requirements. 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 because locking delays are eliminated. However,

during periods of low discharges, the dam must be raised to ensure sufficient depth for navigation. Movable gates which can be traversed may be attached to a sill to form such a dam.

a. Rationale. The primary need for a navigable pass is a dam which provides an area free of piers or other obstructions; therefore, 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 to allow passage of 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 at projects on the Ohio, Ouachita, and Black Rivers. Pass widths vary from 200 ft on the Ouachita and Black Rivers to 932 and 1,248 ft on the Ohio River.

b. Gate types. In addition, the Corps operates dams on the Illinois Waterway at which tows transit the regulating wicket section during higher stages. Gate types usually considered for navigable passes include chanoine wickets, hydraulically operated bottom-hinged gates, and hinged-crest gates. Bottom-hinged wickets are currently being designed for the Olmsted Locks and Dam project on the Ohio River. These wickets are 26 ft long, 9 ft 8 in. wide, and have a 4-in. gap between gates. This project will provide a 2,200-ft-long navigable gated spillway dam which will regulate river flows as required for navigation (see Plates 3 and 4). Descriptions for wicket gates are included under paragraph 5-2*e*. Hinged-crest gates can provide the clear pass area required if designed without piers. Hinged-crest gates are being designed for the Montgomery Point project on the Arkansas River. These gates are nominally 30 ft long and retain a maximum pool differential of about 13 ft. Discussions of hinged-crest gates and drum gates are contained in paragraphs 5-2*b* and 5-2*f*.

c. Additional benefits. Navigable pass gate spillways may provide additional benefits. Because they are generally less massive they are less costly to construct. They are aesthetically pleasing because they are submerged a portion of the time and are less imposing than a
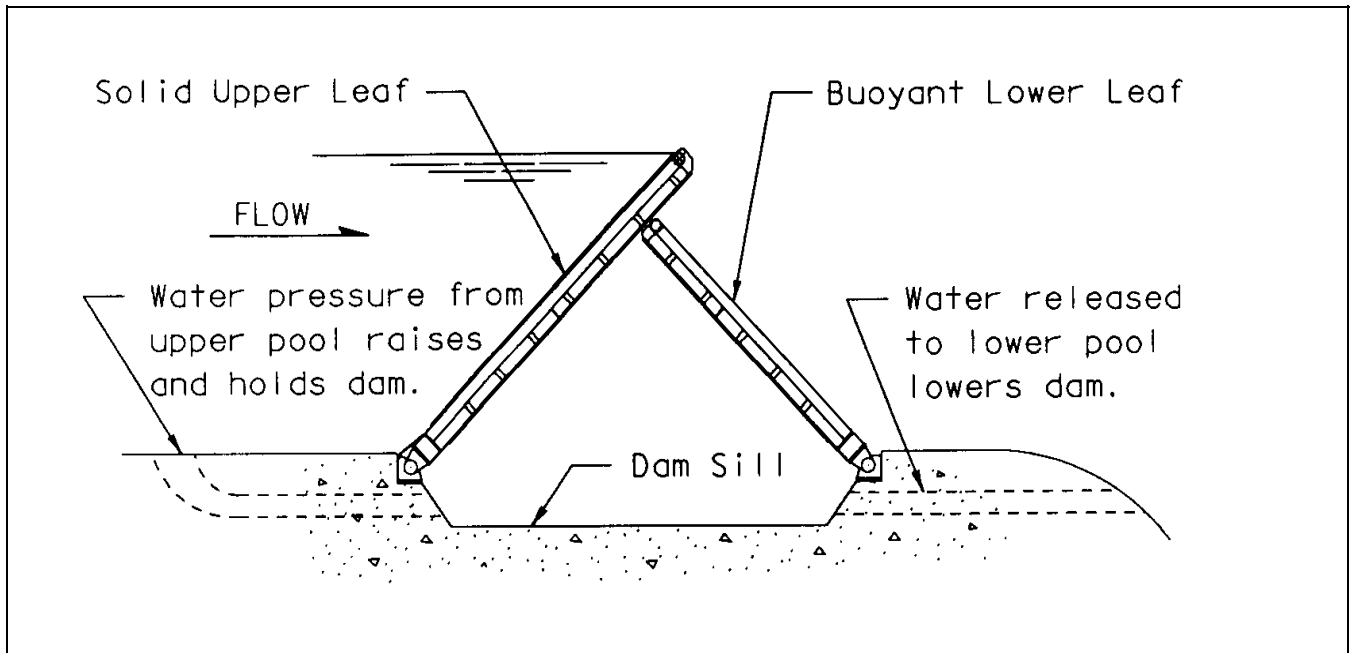
Figure 5-6. Bear trap gate
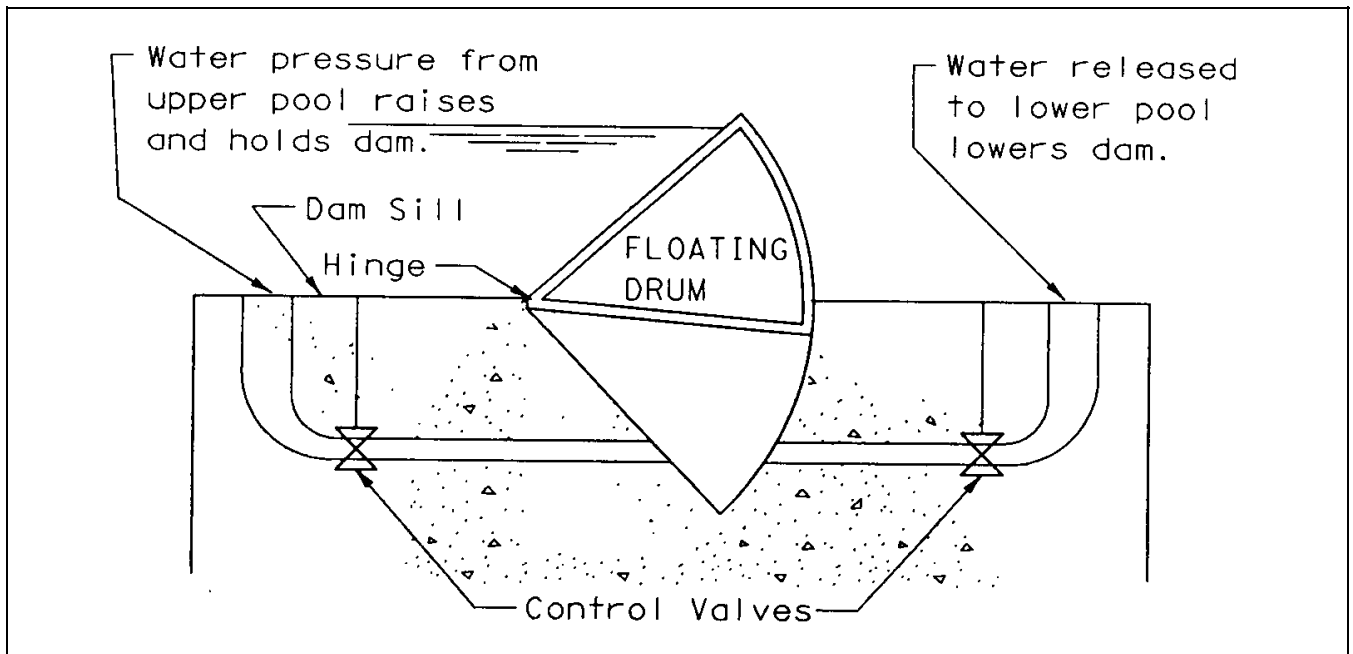
Figure 5-7. Drum gate
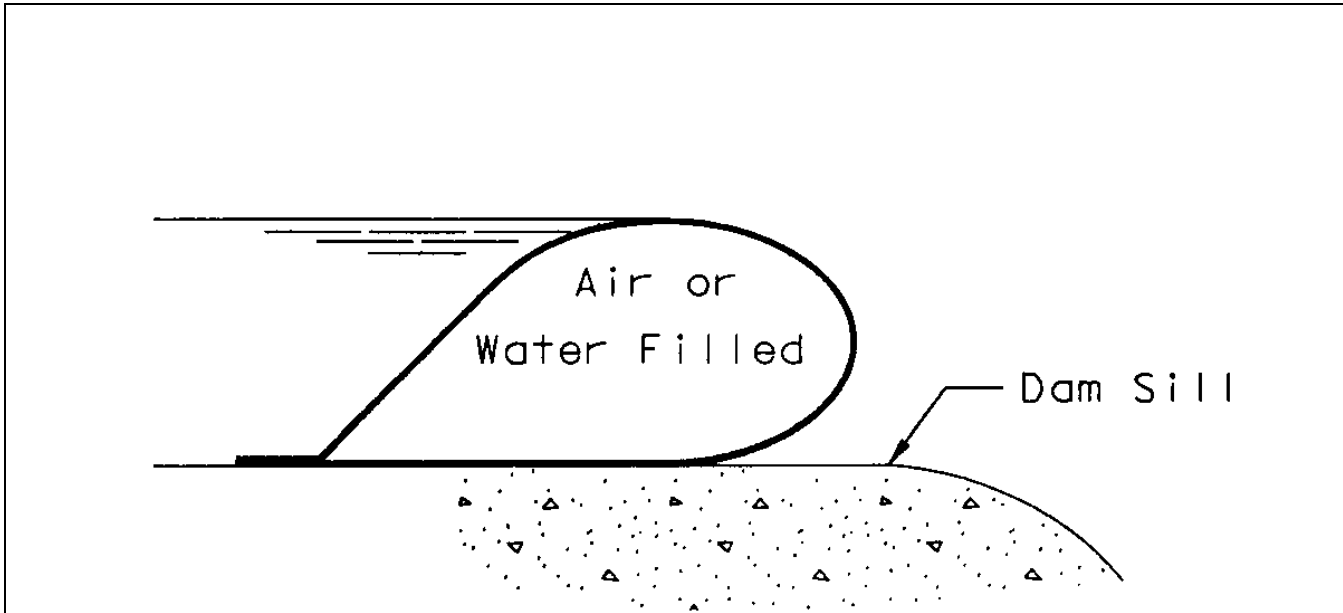
Figure 5-8. Inflatable rubber gate

tainter-gated structure when raised. They may be less likely to be struck by a tow because they are lowered during higher river conditions, and a weak-link protection is easily accommodated for excessive impact. These spillways generally offer more redundancy because the gates are smaller and there are more gates, and it is generally easier to provide them with a backup operating system, such as a normal system which raises the gate with hydraulic cylinders, as well as a boat-operated backup system. They may perform better in areas of high seismicity because of lower structural height and lower mass. They are also more adaptable to multiple operating settings which spread flow over the width of the river or concentrate it. The most important negative consideration is that maintenance of these spillways may be more difficult to accomplish and requires careful planning.

5-4. Fixed Crest

Fixed-crest (fixed-weir) dams are uncontrolled spillways. For overflow structures this spillway can constitute the entire navigation dam or a segment of it. This type of dam is commonly utilized to "tie" gated dam sections into the bank or abutment. The advantage of uncontrolled spillways is their simplicity of both operation and maintenance; the dam structure contains no moving parts or equipment that could be subject to malfunctioning. The toe of the weir is subject to high-velocity, turbulent flows which may necessitate significant scour protection downstream from the dam to preserve the integrity of the

foundation (see EM 1110-2-1605). Additionally, the uncontrolled spillway may raise the flood level of certain frequency floods and may, therefore, require mitigation of this effect. Fixed-crest dams may be navigated, in some instances, during high water events which provide sufficient clearance over the weir. A lock and dam project with only an uncontrolled spillway will usually require higher lock walls than a project with a controlled spillway. An operational disadvantage of navigation projects with uncontrolled spillways is the increased possibility of pleasure boat accidents, because the drop in water surface at the weir is difficult to recognize from upstream. This hazard must be noted with proper warning signs and devices. As riverflows increase, a pool elevation may be reached where project navigation is suspended. In order to mitigate the effect of upstream flooding at uncontrolled spillways, locks are sometimes used as additional floodways by pinning the gates in an open position.

a. Structure types. Fixed-crest weir spillways normally utilized with navigation projects include concrete gravity monoliths, concrete-capped or concrete-filled cellular sheet pile structures, and rock fill dams with a sheet pile cutoff wall incorporated in the fill. Additional types of structures which may be utilized include roller-compacted concrete, reinforced-earth structures, I-walls, T-walls, counterforted or buttressed concrete walls, bin walls, and mechanically stabilized walls. If favorable foundation conditions exist, tied-back or tied-down walls may be appropriate.

b. Crest shape. The shape of the fixed crest is important. Under highly submerged conditions, the shape has little impact on capacity. However, overflow sections having significant head differentials will normally require an ogee-shaped crest, energy dissipation structures, and downstream channel protection. For this reason, many fixed-crest spillways which are combined with gated spillways to form the dam are constructed to a level somewhat above normal pool (2 ft is common), so that tailwater and headwater levels are approximately equal when flow over the crest is initiated. This may allow elimination of any energy dissipation structure.

c. Nonoverflow structures. Fixed-crest dams may also be nonoverflow structures. These structures may be earth or rock fill, cellular, concrete gravity, or any of the above-noted types of walls. For additional information, see EM 1110-2-1902, EM 1110-2-2300, and EM 1110-2-2503. Also, see Plates 10, 15, and 16.

5-5. Piers

a. General. Pier shapes and configurations affect the hydraulic performance and discharge capacity of dams. Piers for dams that have tainter-gated spillways must be wide enough and long enough to accommodate trunnion anchorages and girders, gate operating machinery, stairwells, recesses for unwatering bulkheads, and recesses for second-pour concrete for side seal rubbing plates. Piers for dams that have vertical-lift spillway gates need to be wide enough and long enough to allow for vertical operating recesses and bearing surfaces for the gates, gate operating machinery, gantry cranes, and recesses. Piers for dams with roller spillway gates require widths and lengths to accommodate the operating track (rack), bearing surfaces, gate operating machinery, and recesses for unwatering bulkheads. Spillway bay widths, pier height, and structural design requirements are also controlling factors. Corps dams in existence have pier thicknesses in the range of 8 to 15 ft. For piers, the most common and usually most satisfactory design is a semicircular pier nose shape. EM 1110-2-1605 provides more information on spillway pier configuration.

b. Trunnion girders and anchorages. Most recent tainter-gated dams designed and built by the Corps have prestressed concrete trunnion girders which bear against the downstream face of the pier and provide operational and stationary support for the tainter gates. Usually the girders are of the post-tensioned design. The larger girders are usually cast in place, whereas the smaller ones can be precast and then lifted into place by crane. The girder should be located above most flood elevations. However, submergence is sometimes allowed in the range of 5 to 10 percent of the time. These girders have functioned satisfactorily on many Corps projects with very little maintenance required and only a few instances of nonserious slippage of the anchorages. Structural steel trunnion girders have also been used successfully with the prestressed anchorage system described below.

(1) The trunnion anchorage assembly is composed of a grid of prestressing rods encased in steel pipes to allow for later stressing. The assemblies slope down within the pier toward the upstream face of the pier. Anchor plates are provided at the upstream end of the prestress rods. Bell- or ring-type anchors should not be used because it is difficult to ensure concrete consolidation within these devices. It is possible that the use of these anchorages has been the source of observed anchorage slippage. The assemblies are encased in a zone of high-strength concrete. The downstream end of the rods extends through pipe sleeves in the trunnion girder, and the rods are anchored on the downstream face of the girder. The rods extend beyond the girder to allow attachment of a hydraulic jack for initial stressing and for future jacking to check stress retention. After the initial stressing, a nonhardening compound such as NO-OX-ID is pumped into the annular space between the rod and the pipe sleeve to allow for future restressing of the rods, if it is found necessary. The downstream ends of the rods are coated with a rust-preventing compound and are enclosed in removable covers. Many installations have used steel pipes that have been grouted, in which case the anchors then become bonded anchors. However, in locations where rods are grouted, there is no opportunity to redress design concerns at a later date. If the pipe enclosures are grouted, then the grouting mix should be of a material that does not expose the rods to hydrogen embrittlement.

(2) Pier anchorages are not required for vertical and roller spillway gates because these gates transfer their load into the piers through bearing surfaces in pier recesses.

(3) Figure 5-9 and Plate 8 show a typical prestressed trunnion girder and anchorage. Further guidance for layout and design is contained in EM 1110-2-2702.

c. Finite element modeling of structure anchor forces. The prestressed trunnion girder and pier anchorage should be designed to resist all possible combinations of tainter gate reactions. A conventional beam theory usage will usually be satisfactory for preliminary design of the pier. However, a finite element analysis of a girder and pier section should be used to determine internal
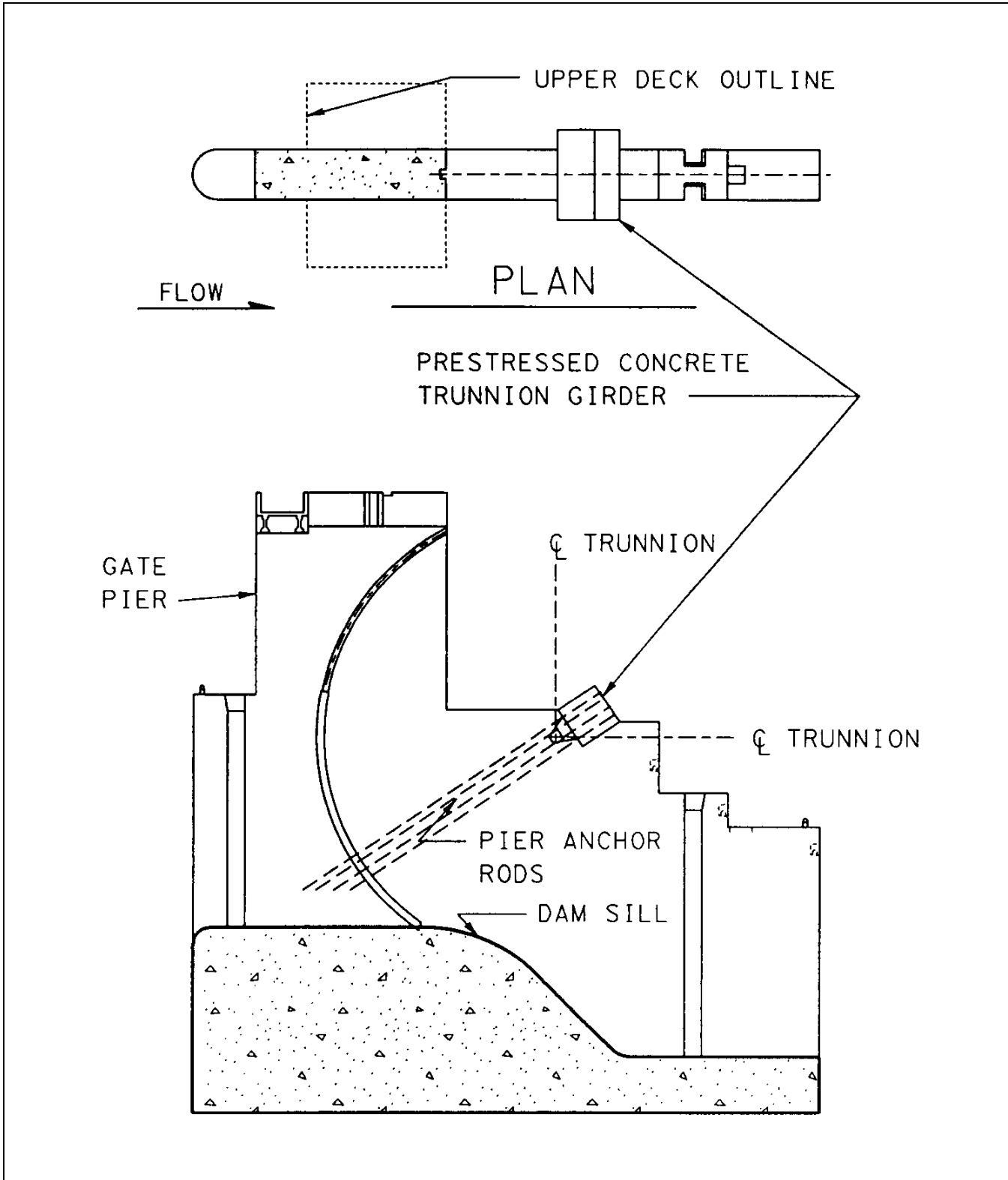
Figure 5-9. Prestressed concrete trunnion girder and anchor rods

stresses in the prestressed or post-tensioned areas. Use of the finite element analysis has shown that girder and pier internal stresses are greater than those resulting from calculations made using the straight-line conventional-beam theory.

d. Operation and fabrication parameters which determine pier dimensions. Pier width, length, and height are based primarily on the following operational features: spillway discharge flow-shape requirements for nose of pier, height of spillway gate in closed position, travel of spillway gate to fully open position, trunnion girder location and trunnion anchorage requirements (for tainter gate), machinery support requirements for spillway gate operation, elevation of service bridge and service bridge supports, recesses (slots) for upstream and downstream maintenance bulkheads, dogging devices for bulkheads, and interior personnel stairwell.

5-6. Miscellaneous Structural Features

a. General. Various monoliths are designed to satisfy hydraulic requirements, maintain foundation stability, provide foundation seepage control, and retain soil where differences in grade elevations exist.

b. Stilling basins. Stilling basins are designed primarily to prevent erosion of foundation materials downstream of the dam, to furnish an acceptable seepage gradient for permeable foundations, and to allow for energy dissipation. Expansion joints separate stilling basin slabs from each other, so each slab acts independently from other slab units. Sheet piles can be located below the stilling basin at the perimeter to prevent piping of foundation material due to seepage pressures. Stilling basin slabs (or spillway aprons) are typically constructed of reinforced concrete; however, roller-compacted concrete (RCC) slabs may be considered where reinforced concrete elements are not required. Stilling basin slabs are designed to withstand uplift loads acting over the stilling basin length. For permeable foundations, the slab thickness must be such that the submerged weight of the concrete is sufficient to overbalance the uplift effect resulting from the increase in static head below the hydraulic jump. Drain holes should be considered for relief of the pressure differential, provided the foundation material will not erode through the drain holes and compromise the stability of the slab. Slabs on rock foundations are typically anchored to the rock with steel bars in a grid configuration. If horizontal bedding planes are present in the foundation rock, the upper rock strata will be subjected to a net upward pressure, and the slab anchorage should be carried to a depth below which the

upward pressure is balanced by the submerged weight of the slab and rock. When energy dissipation is accomplished with the aid of baffle piers and plain or dentated end sills, these structures are typically anchored to the slab and designed for the impact of the water jet and flowing ice or debris. However, such structures are usually at sufficient depth below tailwater to keep them submerged. The hydraulic loading may be estimated from the total pressure on the projected area computed from the maximum expected velocity of the impinging water (see EM 1110-2-1605).

c. Training walls. Training walls are designed to control flows upstream and downstream of the dam where variations in the project features may cause unwanted hydraulic effects. Flows through the dam may produce eddies which cause adverse navigation approach conditions, damage to streambed and slope protection, and sedimentation problems. Training walls are used to direct intake or discharge flows. The elevation of the top of the training walls is normally selected to prevent overtopping at all but the highest discharges. Training walls are normally extended at a constant top elevation to the end of the stilling basin. Adjacent project features and topography have a significant impact on training wall design (see Plate 16). Training walls are typically constructed of reinforced concrete with an inverted "T" cross-section configuration, and are designed to withstand the differential load effects caused by variations in hydraulic profile and the variation in sediment deposits that can occur on each side of the wall. The estimation of these loading conditions can be derived from hydraulic model studies. See EM 1110-2-1603 and EM 1110-2-1605 for determining hydraulic forces (static and dynamic) on stilling basin training walls.

d. Gate pier extensions. In accordance with EM 1110-2-1605, gate pier extensions are required to extend into the basin to a position 5 ft upstream of the baffles to prevent return flow from inoperative bays. The pier extensions can be extended farther downstream if required for stability. These extensions are required to ensure adequate stilling basin performance. The pier extension should be at least 1 ft 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.

e. End sill. A sloping end sill is normally required to spread the flow for single gate operation. This slope is normally 1V on 5H. 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. 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 hydraulic jump downstream.

f. Grade separation walls. Grade separation walls are required where transitions between differing grade elevations cannot be achieved with a stable slope. The wall may also be configured to function as a training wall. Grade separation walls are designed as retaining walls with proper consideration of the fully submerged condition. The required factors of safety are the same as for the navigation lock and dam structures. Sedimentation buildup may cause retained soil loading significantly different from the constructed grades (see Plate 16). Grade separation walls are typically constructed of reinforced concrete with a "T," or retaining wall, configuration. An unreinforced gravity wall RCC may be considered in appropriate cases.

g. Structural separation walls. Walls which separate individual lock structures or which separate a lock structure from the dam gate or abutment structures will vary considerably with the site and type of project selected. The top of the wall will, at minimum, equal the normal upper pool level plus freeboard (1 to 2 ft) but will most generally be equal to the top of the lock walls to allow proper navigation during higher river stages. See Plate 1.

5-7. Special Design Considerations

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

b. Fish passage facilities. Most fish passage facilities are located at the dams on the Columbia and Snake Rivers. These structures are normally ladder type structures, fish screens, sluiceways, etc. See Plate 12 for typical fish ladder locations at a lock and dam project.

c. Ice control methods. It is desirable and often essential to continue operation of navigation dams and spillways during winter (see EM 1110-8-1(FR)). 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. Provisions to move ice past or through dams have been many and varied and none have met with perfect success. At some locations, it is preferable to retain the ice in the upstream pool, while at others an ice-passing capability is necessary. Regulating gates on a dam structure can be used to pass ice and debris by either underflow or overflow. In the first case, the gates are opened sufficiently wide to create enough flow that accumulated ice and debris are pulled from the upper pool to the lower pool, to be carried from the structure by the current. The magnitude of opening for successful operation depends on local conditions and experience; it is usually one-third to fully opened gate, depending on tailwater level. Hydraulic model tests give some indication of the required opening for new structures. One of the dangers of this operation is that it often causes downstream scour holes. To prevent occurrence of scour, proper scour protection and/or energy dissipation must be provided. Spillway openings should be as wide as practicable to minimize arching of ice 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 floes 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, ice may be passed over the tops of gates; however, this has met with only limited success under certain ice and flow conditions. 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 CRREL. EM 1110-2-1612 provides additional information on ice control methods.

Chapter 6
Maintenance and Emergency Closure Facilities

6-1. Maintenance and Emergency Closure

a. General. Accidents involving the dam are usually caused by tows that have "broken up," resulting in barges being lodged in gate bays and/or wrapped around dam piers. Since bulkheads cannot be placed until the damaged barges are removed, and only one gate bay at a time can be closed using emergency closure, the practicability of bulkhead use and corresponding benefits are limited. The consequences of inoperable navigational dam gates may include loss of pool or higher than normal induced stages. Either event may involve significant economic losses. Measures to allow maintenance of dam gates and operating machinery, with analysis of the costs of providing emergency closure and the corresponding benefits (with water potentially flowing uncontrolled through a gate bay area), should be addressed in the design documents. Since access to spillway gates is usually via the closed lock gates, a contingency plan should be developed for access to spillway gates so closure can be made in case of an accident.

b. Spillway capacity. 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 spillway capacity should be sufficient to handle anticipated high flows without causing detrimental increase in upstream stages. If feasible, repair and 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 and equipped for this purpose.

c. Emergency closure. Although emergency-closure bulkheads or vertical-lift gates are normally located upstream of the spillway gates, several instances of barges becoming lodged at the bulkhead location have precluded installation of the emergency closure. Consideration should be given to locations either upstream or downstream of the service gate to provide optimum protection. Potential for serious damage to a navigation dam exists due to the presence of navigation traffic. Appendix C includes descriptions of three major accidents that are representative of what can occur (Markland in 1967 and Maxwell in 1985 and 1990). In the case of collision, damage can vary from inconsequential to major, 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 likely to cause serious damage to the structure in case of collision.

d. Spillway gate positions. For spillway gates, the two positions presenting the least potential for damage at many projects are the fully raised position, particularly if this is higher than barges or tows passing through gate bays, and the fully closed position. A particularly vulnerable position is that of the lips of 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 that the gates be raised to a position above the highest expected water level. Designers may find it prudent to include remote operating capability to permit quick action on the part of operators during emergencies. In 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).

6-2. Maintenance of Gated Nonnavigable Spillway Structures

a. Location. Maintenance of this type of structure is normally accomplished with the combined use of an upstream maintenance or emergency bulkhead and a downstream maintenance bulkhead (if necessary) to allow unwatering the gate (bay) during maintenance activities. Emergency closure equipment should be stored at the dam site. Downstream maintenance closures may be stored at the site or at a central location if used for several dams. Installation can involve the use of hoist cars, cranes, stiff-leg derricks, derrick boats, and in limited instances, divers.

b. Maintenance closure types. Since maintenance closure structures are for use when a spillway bay is to be unwatered for inspection or maintenance, they are designed only for static heads and cannot be installed in flowing water. Appropriate pier recesses and sills must be provided to allow for installation of these closure structures. The maintenance closure structure selected for use at a particular dam will depend on the type of gate and associated piers and sills, whether or not there are bulkheads available for use in the waterway system, what equipment or methods are readily available for use in transporting or installing the bulkheads, and economy and

reliability. The types of maintenance closures that have been used most are sectional bulkheads, dewatering boxes, vertical-lift gates, Poiree dams, and needle dams. Figure 6-1, *a-e*, shows these types of maintenance closures. Poiree dams and needle dams are generally not recommended because of the need to use divers to install them and the length of time required for the installation. Dewatering boxes are suitable only for wicket dams.

(1) Sectional bulkheads. This type of closure structure can be constructed of welded structural steel or riveted aluminum material. The limiting height of each individual section is governed by the handling capacity of the available handling equipment. Lifting beams or other provisions must be included. High-strength steels or aluminum can be used to lessen the weight of these bulkhead sections. Riveted aluminum bulkheads are in use on the 110-ft-wide lower Ohio River tainter gate structures because the bulkheads have to be transported from one dam to another and installed by a limited-lifting-capacity floating crane (derrick boat). These bulkheads are for use only downstream of the tainter gates; they are also used downstream at the locks. The maintenance bulkheads consist of trusses and skin plates or girders and skin plates. Wheels are not furnished on the ends since the bulkheads are installed in static water. The water load on the bulkheads is transferred to the pier walls at the pier wall recesses, and the sill carries only the dead weight of the bulkheads. Sectional bulkheads are used more than any other type of maintenance closure for ease of installation by floating plant.

(2) Poiree dams. Poiree dam closures are constructed of structural steel and are composed of a series of vertical A-frame truss members which are set and pinned by divers into an anchorage casting or shoe embedded in the concrete sill. When these members are positioned and properly stabilized, panels are placed on the upstream face of the A-frames. These panels are usually set by a derrick boat with the help of divers to position the underwater parts. Usually, one Poiree dam will be utilized for several dams in a waterway system. The water load on the Poiree dam is transferred by the A-frames to the concrete sill through the embedded anchorages. The Poiree dam arrangement is generally not selected for new designs because better, safer options are now available.

(3) Needle dams. Needle dams are constructed of structural steel and are composed of a horizontal beam or girder or triangular truss supported by the piers at each end and a series of needles or vertical panels that rest against the support at the top. These needles are positioned by a diver into the proper location on the concrete

sill at the bottom. Thus, a portion of the load is transferred into the pier recesses at the top, and a portion of the load goes into the sill from each individual needle or panel. A pipe or similar object is positioned at each of the abutting edges of the vertical panels to better seal the opening. For further watertightness, cinders are sometimes used. This type of closure is more suitable for narrow gates because the needle beam weight and configuration become limiting factors. Needle dams are adaptable for either upstream or downstream closures. As with the Poiree dam, this arrangement is generally not used for new designs.

(4) Floating closure. Floating closure structures have not been used extensively, but where they have been used they have functioned well. One such caisson is used for locks on the Columbia and Snake River systems. Another floating caisson is in use for 110-ft-wide locks on the Tennessee and Cumberland River Waterways. The structural steel caissons are composed of watertight compartments which allow the towing of the floating caisson from one lock to another. The compartments are filled with water so that the caisson can be positioned in recesses in the lock wall and sunk in place across the lock chamber. Compressed air is used to remove the water from the caisson and return it to floating capability. In view of the success of these facilities, a floating caisson is feasible.

(5) Braced-box cofferdam. A large braced-box cofferdam, constructed and installed as two halves (to facilitate use of an existing 100-ton derrick boat), was successfully utilized to repair the Ohio River Lock and Dam 52 temporary lock (cellular/concrete) miter gate sill.

(6) Use of emergency closures for maintenance closures. At dams where an emergency closure structure is provided upstream of the service gate, the emergency structure can also be used for maintenance unwatering. Additionally, recesses can be provided in the piers for installation of sectional bulkheads similar to the ones used for unwatering maintenance downstream of the gate.

6-3. Emergency Closure of Gated, Nonnavigable Spillway Structures

Where a large level of damage could result from water flowing free through a dam gate bay, an emergency means to quickly stop the unrestricted flow should be studied. This unrestricted flow could happen if the gates were rendered inoperable or knocked out due to being rammed by a tow.
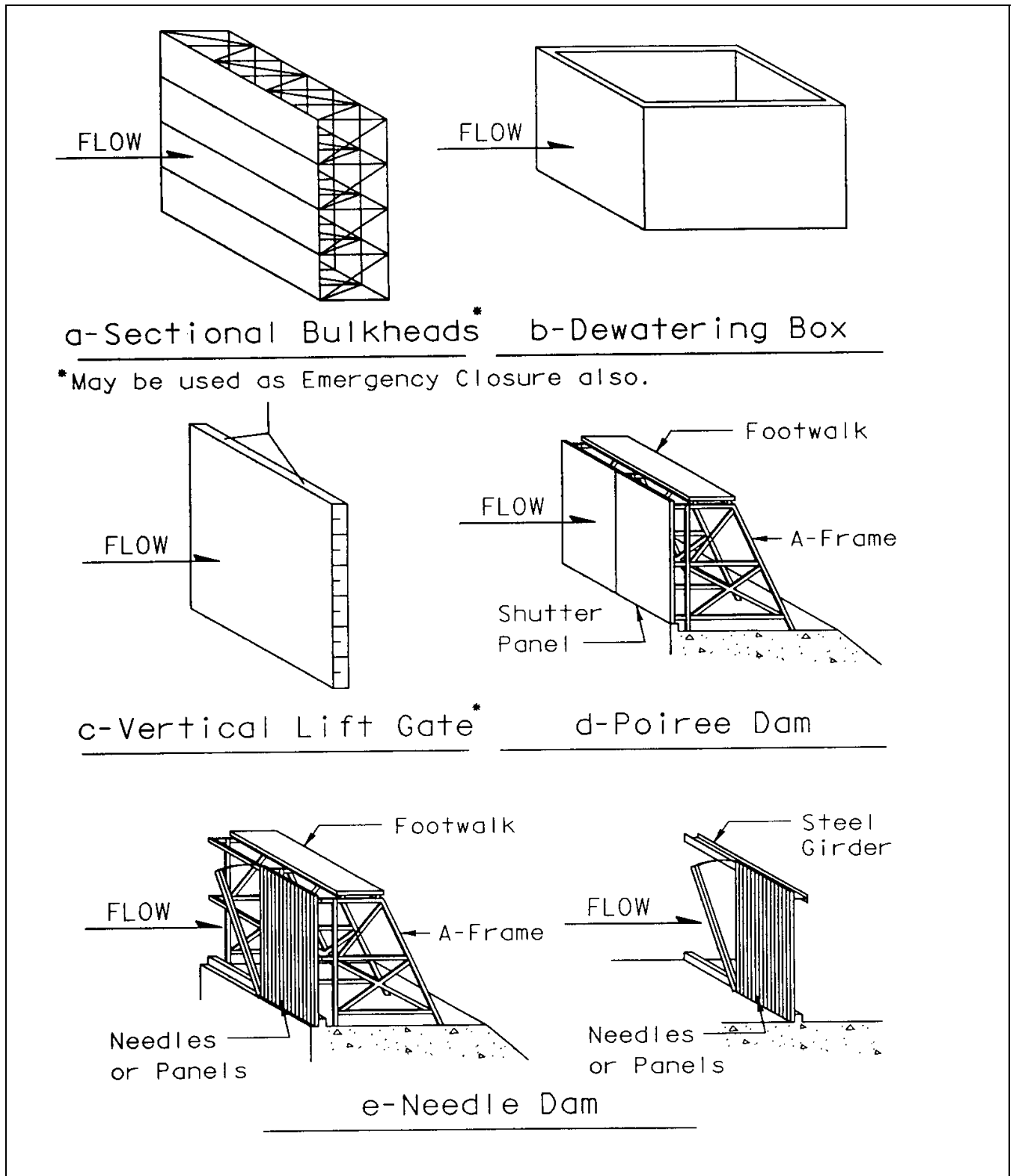
Figure 6-1. Maintenance closures

a. General. The following conditions must be considered and evaluated in determining whether or not an emergency closure structure is necessary: consequences of loss of pool (such as effects on water intake and outfall structures and docks and towing industry losses); economic losses to shipping interests due to halt of river traffic; possible flood damage and danger to people downstream; and consequences to channel banks in pool due to sudden drawdown. If incorporation of emergency closure is found to be economically justified, all elements of the emergency closure structure, including the handling equipment and machinery and the structure itself, should be ready for use 24 hours a day. Proper maintenance of all elements is necessary, along with periodic practice installation of the closure. Reliability and a fast installation time are a must. The types of emergency closures that have been used are stacked bulkheads and vertical-lift gates.

b. Overhead stacked bulkheads.

(1) General. The stacked bulkhead arrangement has proven to be the most dependable and reliable for emergency closure purposes. Most major dams built in recent years have utilized this concept successfully. The sectional bulkheads (stop logs) have end rollers and are made up of trusses and skin plates. The lifting and installation equipment is usually very expensive and consists of either a bridge crane (hoist car) or a traveling gantry crane.

(2) Bridge crane (hoist car). The bridge crane (hoist car) is sometimes referred to as a locomotive crane. See Plate 9. Most of the tainter-gated dams on the Ohio River and the new Melvin Price Locks and Dam on the Mississippi River use a bridge crane for handling the bulkheads. At many of these projects, the spillway bays and the lock width are both 110 ft wide so that the emergency bulkheads can be used to close off either an individual 110-ft-wide spillway bay or one 110-ft-wide lock chamber. Thus, the service bridge is laid out so that it and the bridge crane and bulkheads serve both the locks and dam spillways. An auxiliary crane is located on top of the bridge crane for use in handling small loads during maintenance or unwatering activities.

(3) Detailed bulkhead installation procedure. Past experience and model testing by WES have shown that bulkheads cannot be lowered safely one at a time in flowing water. Therefore, the stacked bulkhead system was developed so that the flowing water never goes over the top of the bulkheads.

(a) The Ohio River and Mississippi River Dams mentioned above have a hoist car mounted on rails on a service bridge which serves both the dam spillway and the locks. Normally, the bulkheads are stored singly over the spillway bays on retractable dogging devices on the service bridge piers. The required lifting beam will be stored on one of the dogged bulkheads. The hoist car can travel the full length of the service bridge but can carry only the lifting beam and one bulkhead while traveling.

(b) The following example details the installation procedure for three bulkheads; however, the same sequence of activities would be true for any number of bulkheads. The hoist car will pick up one bulkhead with a lifting beam and move it to the location for unwatering; at this location it will be placed on the dogging device. The hoist car will then move with the lifting beam and pick up another bulkhead and move back and place it atop and latch it to the first one. The two will then be raised enough to retract and reposition the dogging device so that they can be lowered onto it. The hoist car moves with the lifting beam to get a third bulkhead, which is placed and latched to the other two, and then lifts the three bulkheads and lowers them as a unit into the flowing water after the dogging device has been retracted. In this manner, the flowing water is stopped without being allowed to flow over the top of any of the bulkheads. After the bulkheads have served their purpose and the spillway gate is closed, the bulkheads are removed one at a time in a balanced (static) pool situation by the hoist car and returned to their individually dogged (stored) position.

c. Vertical-lift gates. Vertical-lift gates may also be utilized as an emergency closure--especially on dams that have vertical-lift spillway service gates which are installed and removed by a traveling gantry crane. These gates would be similar to the vertical-lift service gates described in paragraph 5-2*c*.

6-4. Maintenance of Gated, Navigable Spillways

Maintenance of gated navigable spillways must be accomplished under somewhat different conditions than that of nonnavigable spillways. The absence of intermediate piers generally limits the type of closure to floating bulkheads, rigid-box closures, Poiree dams, separate duplicate gates, collapsible A-Frame, and, for low-head applications, removable-post and stop-log installations. The gated navigable pass spillway is generally less vulnerable to navigation accidents than nonnavigable spillways because the gates are lowered during high water events,

which produce conditions most likely to lead to an accident.

Also, since a damaged navigable pass gate is usually comparatively small, and redundant operating procedures are easily accommodated, an emergency gate for this type of spillway is generally excluded.

a. Machinery. Machinery for gated navigable spillways, which can be incorporated in a dry gallery in the dam sill, is much easier to maintain than machinery for nonnavigable spillways. Also, because navigable pass gates are generally smaller, gate and operator units (cylinders) which can be easily removed and replaced by spare units are desirable. This shortens exposure time and lessens the difficulty of normal maintenance by allowing offsite rehabilitation of the major units. Use of various caissons, floating bulkheads, and gates for maintenance of gated, navigable spillways is similar to that described for nonnavigable spillways, with the exception that spillway piers are absent. The procedure utilized for maintenance will rely on a floating plant, which must be included in the maintenance scheme unless duplicate gates are used as backup to the service gates.

b. Procedure. At the Olmsted Locks and Dam project, the procedure for performing maintenance will be accomplished with the use of a one-piece shutter box, which allows work on one, two, or three gates at one setting. The upstream shutter box will be placed any time work is required on a wicket. With the box in place, any "in-the-wet" work can be done. The water level within the box will be the lower pool level. A brace will be provided to prop a wicket vertically, if desired. The downstream shutter box is required when it is necessary to dewater the area. At the top, a truss may be provided to carry the loading.

c. Maintenance. Maintenance of the hydraulic system will be performed by removing the entire hydraulic system (cylinders, flexible lines, bearings, cover seal, rubber boot, etc.) from the precast frame in the sill. The unit will be removed by personnel on the work boat and a new or refurbished one inserted. This will allow the unit to be refurbished for future use in the maintenance shop as time permits. The hydraulic pumps and valves will have shutoff valves and unions to allow removal of any component from the dry gallery. The hydraulic lines which penetrate the gallery walls will be located inside a sleeve for replacement when the hydraulic cylinders are removed. The wickets will be pinned in place, and the hurters will be bolted down. These items will be removed by the personnel on the work boat.

6-5. Emergency Closure of a Gated, Navigable Spillway

As stated earlier, if flow conditions and redundant (backup) systems of operation are incorporated (and loss of pool is a low-risk occurrence), then emergency closure may not be required. An example of this is included in the Olmsted project. At this project, all of the dam wicket gates will be required to be up only during a very infrequent low-flow event. Damage to a significant portion of the dam can be tolerated (temporarily) by shifting regulation from damaged gates to unused (down) gates. Risk of loss of pool is low, and repair may then be accomplished with more favorable upper and lower pool conditions. Additionally, the wicket gate operating systems may be switched over to the adjacent bank of pumps and motors in the event of failure of one system.

a. Crane operation. The wickets are also designed to be raised and lowered by a crane located on a work boat. In addition to raising and lowering a wicket if the hydraulic system is inoperable, the crane will cut away large debris that collects on the dam by means of a hydraulic shear attachment on its boom, set the shutter boxes, and remove components of the navigable pass such as wickets, cylinders, and hurters. The wickets will use the same basic principle of prop and hurter mechanism used at Locks and Dams 52 and 53 on the Ohio River. The major addition to the prop is the installation of a "weak link" to be used to help prevent serious damage to the wicket during a navigational accident. The prop will be designed to withstand ice and impact loading but to fail and allow the wicket to fall when impacted by a load slightly below the yield strength of the wicket.

b. Further requirements. If emergency closure of a gated navigable pass spillway is required, a floating bulkhead structure with adequate guide and anchorage must be provided, or a separate set of gates must be provided with the appropriate lengthening of the dam sill to accommodate these gates (which are generally upstream of the service gates).

6-6. Maintenance and Emergency Closure of a Fixed-Crest Spillway

Special provisions for fixed-crest spillways are not normally required because of the low risk and low maintenance requirements.

6-7. Floating Plant

a. The floating plant necessary for installing the upstream and downstream spillway gate bay unwatering bulkheads for either maintenance or emergency purposes may be composed of the following:

(1) Derrick boat with suitable capacity and reach.

(2) Work barge to transport sectional bulkheads from storage location to dam spillway.

(3) Work (shop) barge with all necessary equipment such as anchor lines, tools, paint, ladders, sand blasting equipment, safety harness, etc.

(4) Personnel barge with clothing-change facilities for special body covering and footwear plus eating and break accommodations, if not provided on shop barge.

(5) Small boat with outboard motor for transporting personnel to and from bank to work barges and personnel barge.

(6) Work flat (approx. 8 ft × 12 ft).

b. To stabilize all the above floating plant vessels in suitable locations above and below the dam in flowing or quiet water situations, it is necessary to furnish mooring rings, check posts, and line hooks on each pier, and armored mooring holes through the piers to use to anchor lines from the floating plant vessels. For emergency situations, such as free flow through a spillway bay when a spillway gate cannot be closed for some reason, it will be necessary to provide additional anchorage facilities upstream of the dam. These facilities can include deadmen, mooring cells, or pylons on the bank or in the water, and mooring posts on the upper lock wall--all to accommodate floating plant anchor lines.

6-8. Galleries, Adits, and Openings

Normal periodic maintenance and inspection of spillway gates, when the gates are not unwatered, can require providing a means for operating personnel to access certain parts of the gate. This is particularly true for a tainter gate. The dam pier will usually contain a stairwell extending from the service bridge level down to the trunnion girder level. An opening from this stairwell to the trunnion girder on the downstream face of the pier will provide personnel access for greasing the trunnion pin and for inspecting the trunnion anchorage and the trunnion girder. Some tainter gate arms also have handrails for personnel use in accessing the gate body. Also, an access opening may be provided from the stairwell to the spillway face of the pier for wire rope greasing and inspection and gate inspection. For double-skin plate gates and gates with cover plates, this type of opening can allow access to the interior of the gate.

Chapter 7
Seepage Control Measures and Features

7-1. General

Unless properly treated, seepage of reservoir water through a dam foundation and abutments can present serious problems regardless of the type of foundation--rock, soil, or piling. The primary purpose of controlling the seepage of water through a dam foundation is to prevent the foundation material from piping and washing away, which could result in structural failure due to loss of support. A secondary purpose of controlling seepage is to reduce foundation uplift pressures. Impervious (clay) blankets and cutoff walls, in particular, lengthen the seepage path and thereby reduce uplift pressures. However, where foundation drains are provided, it has been Corps practice in the design for sliding and overturning stability to either disregard the benefit of the drains or consider the drains only partially effective in limiting uplift pressures. However, maximum possible uplift values should be used for checking foundation pressures and relief of pile loadings. Piezometers have been used very effectively in some cases to identify problems with existing seepage control systems. When filters are placed under a dam to relieve excess uplift pressures, they are of particular benefit to the stilling basin slab design because the slab without the filter would need to be anchored or of massive thickness. To attenuate, control, collect, and/or direct the seepage discharge, careful and thorough geotechnical and hydraulic studies and evaluations must be made, and proper cutoff (control) features must be designed.

a. Foundation seepage. The particular seepage problem for soil foundations and pile foundations--as well as some rock foundations which have solution channels, rock jointing, and cavities--involves piping out the foundation material. To counteract this piping, seepage control can be accomplished by a site-specific cutoff method featuring foundation grouting and drainage, steel sheet piling cutoff walls, impervious cutoff walls (trenches), concrete cutoff walls, slurry trench cutoffs, or an upstream impervious blanket.

b. Abutment seepage. To prevent the bank from failing and the stream from possibly bypassing the dam, treatment of the dam abutments should include cutoff walls, a competent bank tie-in structure, and bank slope protection.

c. Seepage analysis. Seepage analysis will generally be required. Detailed information on seepage analysis and layout details of seepage control systems are contained in EM 1110-2-1901.

d. References. Detailed information on foundation grouting, planning, and specification writing is contained in EM 1110-2-3506 and in Guide Specifications for Civil Works Construction.

7-2. Foundation Grouting and Drainage

a. Dams founded on rock.

(1) It is customary to grout and drain the foundation rock of concrete gravity dams. This practice works well for defective as well as sound formations. A well-planned, well-executed grouting program will not only reduce seepage through the rock but may also disclose the presence of unsuspected weaknesses in the foundation, thus improving any existing defects. Such a program, therefore, provides an added safety measure and ensures against future trouble.

(2) The most common design consists of a single line of grout holes located near the upstream face of the dam, drilled at five-foot centers and to a depth ranging from four-tenths to six-tenths of the maximum hydrostatic head on the base of the dam. A corresponding line of drainage holes is drilled a few feet downstream from the grout curtain and to a depth roughly two-thirds to three-quarters that of the cutoff curtain. This grout curtain may be constructed by drilling and grouting from a gallery within the dam, from the top of a specified thickness of concrete, or from the top of foundation rock. If a gallery is provided, then a series of drain holes will be drilled from the gallery and located just downstream of the grout curtain. See Figure 7-1 for layout and details of a grouting gallery with foundation grouting holes and foundation drain holes. It is essential to control the grouting pressures so that splitting and lifting of rock will not occur. Thin-bedded rocks are especially susceptible to damage by excessive grout pressures.

(3) When a stilling basin (also referred to as an "apron" or "bucket") is founded on rock, drain holes should be provided in the rock with a collector and discharge system at the founding level for partial relief of the pressure differential. A typical stilling basin
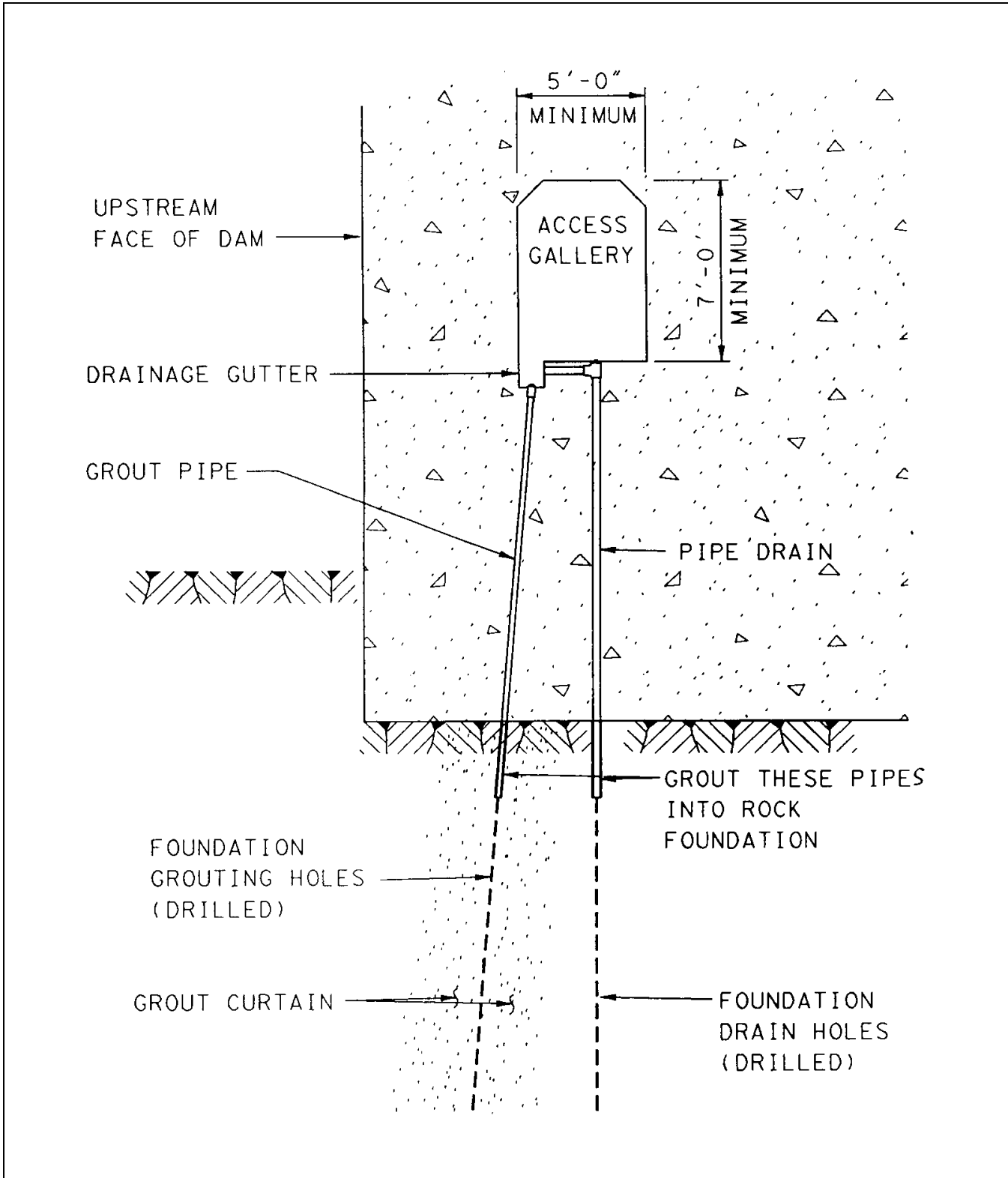
Figure 7-1. Typical grouting and drainage curtain

foundation drainage system and anchorage layout are shown in Figure 7-2.

b. Dams founded on soil or piling. Soil or pile foundations should have upstream and downstream cutoff walls (usually steel sheet piling) under the structure proper plus a cutoff wall underneath the downstream end of the stilling basin. These cutoff walls prevent piping of the foundation material due to seepage pressure. Every effort should be made to provide a reliable drainage system underneath the dam proper and the stilling basin to ensure pressure relief between the cutoffs. Plate 2 shows details of an underdrain system for soil-founded Dam No. 4 on the Red River in Louisiana (Vicksburg district). This system uses 6-in.-diam stainless steel well screen encased in a 2-ft-thick filter material, combined with access manholes, for relief of excess foundation uplift pressures which build up in the foundation. PVC (polyvinyl chloride) pipes connect the manholes to equalize the pressure in the system, and gate valves are provided so that the manholes can be unwatered to allow maintenance of the system.

7-3. Impervious Cutoff Walls (Trenches)

These compacted backfill trenches are constructed from the base of an earth dam structure through the upper pervious layers down to impervious soil layers or to bedrock. The cutoff trench is an extension of the impervious zone of the dam proper. See Plate 10 for details.

7-4. Concrete Cutoff Walls

a. Shallow cutoffs. Shallow depth cutoffs may be constructed where shallow trenches can be excavated and backfilled with concrete. This cutoff method can be applied when navigation dam piers are spaced at a clear distance between piers as wide as the navigation lock. In this case, a dam pier founded on rock may not require a transverse width (for stability) as wide as the pier spacing, and a segment of the spillway section could span between the pier bases. The spillway segment could be supported by the dam pier bases instead of being founded on rock, if overburden material exists. A concrete cutoff wall extending from the spillway segment base down to rock may be feasible. See Plate 8.

b. Intermediate depth cutoffs. An intermediate depth (up to about 80 ft) cutoff can be achieved by the slurry trench method. As the trench is excavated by conventional equipment -- backhoe, dragline, etc. -- the bentonite slurry is introduced to support the sides of the excavation. When the excavation reaches the design depth, impervious material is placed under controlled conditions to establish the cutoff. For major, permanent structures, the slurry should generally be displaced with tremie concrete.

c. Deep cutoffs. When concrete cutoffs need to extend to a great depth below either a concrete or an earth dam structure, a 2.5- to 3-ft-wide excavation is drilled by special equipment and the excavation walls supported by bentonite slurry, which is displaced when the concrete is placed by the tremie method.

7-5. Sheet Pile Cutoff Walls

Continuous steel sheet piling cutoff walls are used for soil-supported and pile-supported concrete dam structures. The piling should be used upstream and usually also downstream and should be embedded 1 to 2 ft into the base of the concrete at the top. The piling will be driven to a depth that satisfies the seepage cutoff requirements indicated by analysis. See Plate 2 for a typical steel sheet pile cutoff. The following discussion will provide guidance for the use of cold-formed, Z-type steel sheet piling as an alternative to hot-rolled, Z-type steel sheet piling. This type of piling is used for straight wall installations where beam strength (bending) is a primary design consideration. While the sheets are required to interlock, no supplier has ever warranted a specific interlock tension value. The hot-rolled piling and the cold-formed piling have markedly different interlock configurations, as depicted in Figure 7-3.

a. Section stability. While neither the hot-rolled nor the cold-formed shapes have been sized to meet any width-depth or width-thickness criteria, it appears that the configuration of the cold-formed sections results in section instability. Currently, there exist no criteria for section stability; therefore, in applications where high stresses are expected, appropriate section stability checks should be made.

b. Seepage. The loose fit of the cold-formed sheets can result in greater seepage than occurs with hot-rolled. However, with time, the interlocks may silt-in and provide adequate control of seepage. Analyses considering the magnitude of seepage and the potential for silting-in should be performed to determine if the cold-formed sheets are acceptable.

c. Installation. Problems have been reported regarding the tendency to drive the cold-formed sheets out of interlocks in areas of hard driving. Also, due to the loose fit, vertical plumb in the plane of the wall is difficult to maintain.
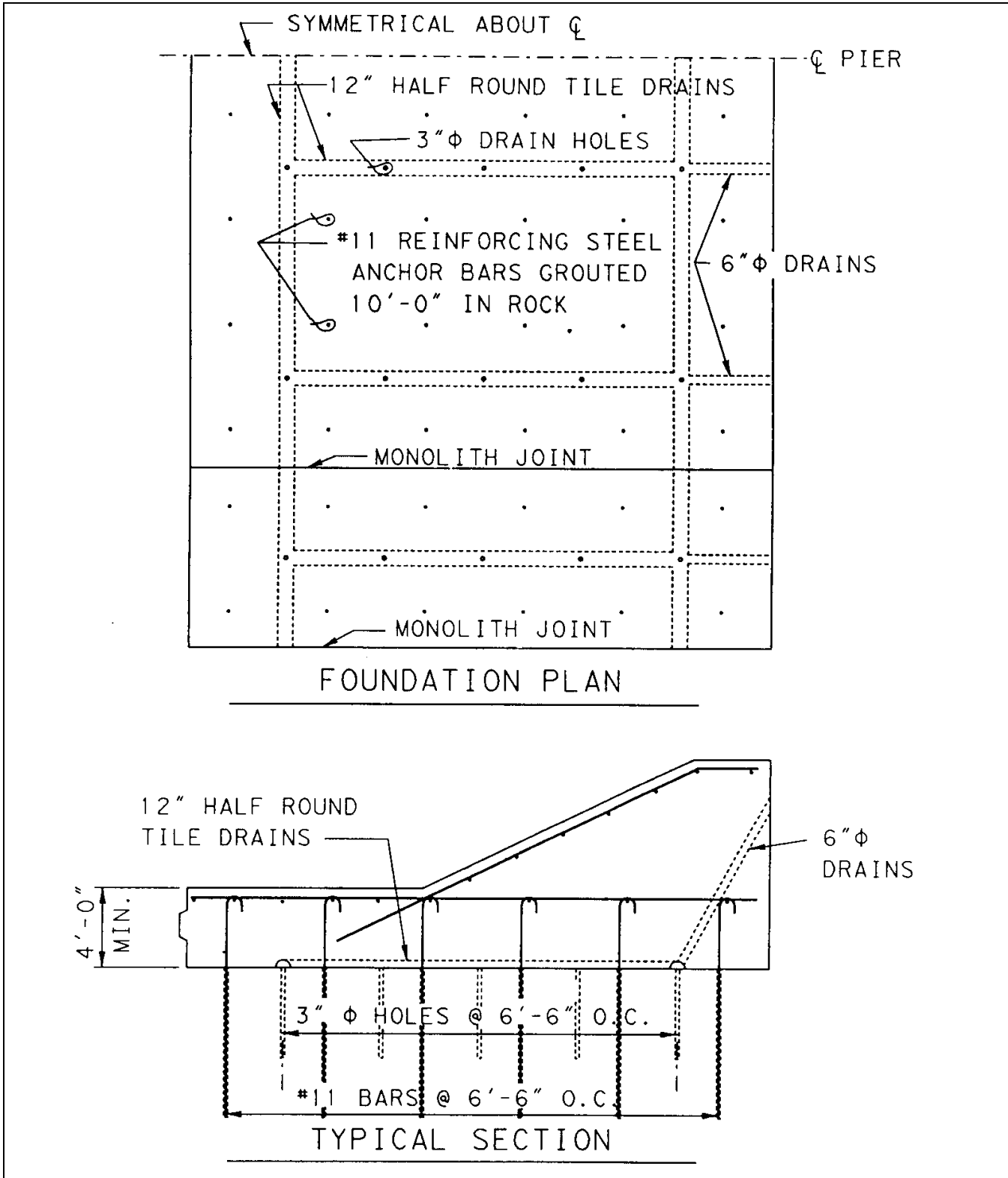
Figure 7-2. Stilling basin drainage and anchorage

$$S = 30.2 \ IN^3/LIN. \ FT. \ OF \ WALL \ (PZ \ 27)$$

HOT ROLLED STEEL SHEET PILING

$$S = 31.6 \ IN^3/LIN. \ FT. \ OF \ WALL \ (CZ \ 114)$$

COLD FORMED STEEL SHEET PILING
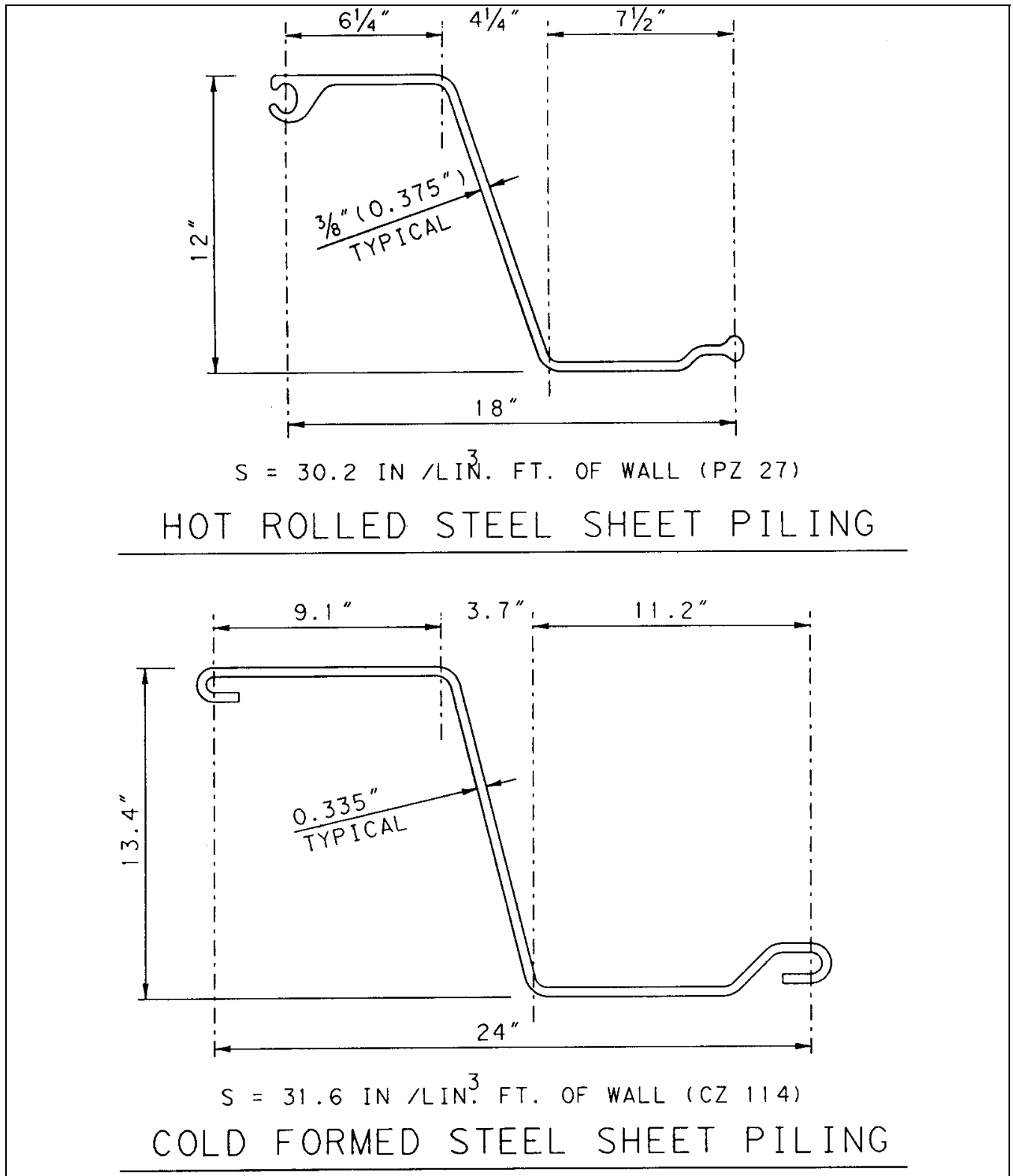
Figure 7-3. Steel sheet piling

d. Corps of Engineers specifications. Guide Specification CW-02411 allows for the use of heavy-gauge cold-formed piling as an alternative to hot-rolled steel sheet piling but gives the following recommendations:

(1) Hot-rolled steel sheet piling sections. Hot-rolled steel sheet piling sections are suitable for applications where interlocked joint strength in tension or section stability is a primary design requirement. Section stability (biaxial stress) is a consideration in highly stressed applications only.

(2) Hot-rolled, light-duty steel sheet piling sections. Hot-rolled, light-duty steel sheet piling sections and cold-formed steel sheet piling sections are suitable for average-depth applications such as trench sheeting and bulkheads in moderate water depths. They are not suitable for applications where they are subjected to high concentrated wale loads or where interlocked joint strength in tension or section stability is a primary design requirement.

(3) Cold-formed, light gauge steel sheet piling sections. Cold-formed, light gauge steel sheet piling sections are suitable for applications with a required minimum sheeting thickness of 0.250 in. or less, low bending and corrosion resistances, and minimal required interlocked joint strength in tension. The corrosion resistance of light gauge sheet piling can be increased by applying a protective coating.

e. Conclusions. Since the use of a different type of piling will depend on site-specific conditions, each Corps office needs to make its own decision on whether or not to use the cold-formed steel sheet piling. The decision should be based on site-specific foundation data, design requirements, and the importance of obtaining the best possible "in-place" assembly to prevent excessive amounts of water from passing through the interlocks. In locations where the piling is installed in a pure bending design requirement and where excessive flow of water through the interlocks with undesirable displacement of material is not a controlling factor, the cold-formed piling should perform equally as well as the hot-rolled piling (assuming the equivalent section properties).

7-6. Upstream Impervious Blanket

An upstream impervious blanket will frequently be advantageous for soil and pile founded dams, as it increases the length of the seepage paths and thereby reduces uplift pressures under the structures and the potential for seepage problems. The blanket will usually be of clay material (rather than concrete) for purposes of economy. An impervious membrane, in lieu of a clay blanket, is not recommended because of the high risk of punctures and tears. Typically, the blanket will be a minimum of 5 ft in thickness, and will extend upstream as necessary for seepage control and extend as necessary to the adjacent channel slopes. To account for potential separation between the blanket and structure, an impervious membrane, with laps to allow lateral movement without tearing the membrane, is attached to the face of the structure and embedded into the blanket. Refer to Plate 11 for details.

Chapter 8
Channel Protection

8-1. General

Stabilization of banks and protection of the stream bed will usually be required for the channel at the dam site. In addition, channel realignment and/or training structures may be required. The specific types of channel protection and stabilization will depend on water velocities, wash from waves, water level fluctuations, soil characteristics, stream alignment, sediment transport, wind velocities, and navigation approach conditions. This chapter will outline, in general terms, the overall requirements for channel protection. More details and guidance are contained in EM 1110-2-1611.

8-2. Erodible Slopes and Stream Bed

a. Erodible slopes. When geotechnical investigations and studies indicate that the stream banks at the dam site will not be stable under project operating conditions, it will be necessary to protect and stabilize these slopes so that they will not erode and displace into the stream or fail internally.

b. Stream bed. The stream bed immediately upstream and downstream of a concrete dam will require protection to prevent displacement of bed material that could undermine the structures and result in failure of the dam and/or appurtenant structures.

c. Scour protection during construction. Each construction scheme must be carefully analyzed to ensure that scour protection is provided where necessary. Successful protection has consisted of timber (lumber) mattresses or riprap, both with and without filter blankets, depending on the soil types and flow conditions. Physical and numerical models have been useful in developing scour protection designs. For cofferdams within the river channel, the riverward corner of the cofferdam is usually the critical point of scour potential. Wing extensions are sometimes provided to minimize the scour potential.

8-3. Typical Materials

a. Slope protection materials. For channel slope protection, the following materials are used most often: riprap stone placed on either a sand-and-gravel filter blanket or a filter cloth, soil-cement paving, concrete paving, steel pile cells filled with stone, and articulated concrete mattresses. To be effective, the slope protection system

material must be sized with a designed thickness and must be provided with drainage blankets or pipes so that water can pass through the protection without displacing the underlying support material. In most cases, the type of material chosen for slope protection will depend on the availability of suitable natural material in the project area, which is likely to be least costly.

b. Streambed protection materials. For streambed protection, the following materials are used: graded stone, derrick stone, concrete paving, trench-filled (stone) revetment, and lumber mattress. The materials chosen for streambed protection will depend on their availability and on specific design and maintenance requirements during the life of the project.

8-4. Dikes

Dikes for bank protection and stabilization can consist of stone, timber pile clusters, or piling with stone fill. These dikes are designed to divert currents away from the bank or improve the alignment and velocity of the currents along the bank. Training dikes can be very beneficial in controlling sediment deposition in the upstream and downstream lock approaches and in diverting spillway discharges away from the lower lock approach. See Plate 7. Wing dikes (also referred to as wing dams or spur dikes) placed approximately normal to the channel or lock approach have proven to be suitable for these purposes. Various types of dikes are described in EM 1110-2-1611.

8-5. Upstream Channel

a. Channel slopes. Just as the bank slope must be designed for stability, so the slope surfaces must be protected. Stone riprap on a sand-and-gravel blanket or on filter cloth can be used. Other forms of slope protection, such as articulated concrete mats, concrete paving, and soil-cement blankets, have also been used. The type of protection material chosen will depend on economy of usage and the ability of the material to satisfy design requirements for the life of the project with minimum maintenance costs.

b. Streambed. A low-head navigation dam will usually require streambed protection upstream of the spillway for a minimum distance equal to the head on the spillway crest. The protection is usually stone; however, concrete aprons have been used. Refer to EM 1110-2-1605 for details. High-head dams will not usually require upstream streambed protection because of the height of the spillway crest above the streambed. The upstream

streambed of a navigation dam with a ported upper guide wall adjacent to the dam spillway will require stone protection at the ported openings to prevent scour of the bed material due to water velocities caused by spillway discharges. This bed protection is especially important for soil-supported, pile-supported, or caisson-supported guide walls. Refer to Plates 3, 5, and 17.

8-6. Downstream Channel

a. Channel slopes. Treatment and protection of downstream channel slopes will be similar to that for upstream channel slopes. However, the thicknesses and the extent of this protection will usually be determined by using the results of the hydraulic model test. Refer to Plates 3 and 5 for more details.

b. Design conditions for streambed downstream of gated spillway. The streambed area immediately downstream of the stilling basin will require a special hydraulic investigation to determine the amount and extent of streambed scour protection needed to counteract the forces created by the spillway discharges. The spillway gate operation design conditions are outlined in ER 1110-2-1458. For a streambed composed of sound rock, no protection may be needed. However, for a readily erodible streambed, several extensive layers of graded rock may be required. Gradation layers, size of stones, and extent of protection are usually designed by hydraulic engineers using guidance contained in EM 1110-2-1605. EM 1110-2-1605 covers scour protection experiences at existing Corps projects and repair recommendations for these projects. EM 1110-2-1601 and EM 1110-2-1901 contain further design information relative to riprap and filter designs.

Chapter 9
Loads

9-1. General

This chapter contains discussion and guidance on loads that can normally be expected to be imparted to a dam or appurtenant structure. Loads described in this section are those resulting from construction activity: lateral earth pressures; hydrostatic, including uplift and hydrostatic pressures; water above or contained within a structure; dynamic forces from impact or seismic activity; line pull forces; wind, wave, ice, and debris forces; gate, bridge, crane, and bulkhead forces; pressures on sheet pile cutoff walls; forces in monolith joints; superstructure weight; and thermal stresses.

9-2. Construction Loads

a. Cofferdam tie-in loads. Cofferdam tie-in loads are encountered on dam projects that are constructed either within the river in phases or where additional dam structures are provided at an existing project. These loads are caused by either cellular or embankment cofferdams constructed for a phase subsequent to the dam construction which uses a portion of the newly completed or existing structure as a portion of the dewatering cofferdam. Loads not to be overlooked in the design of a cofferdam tie-in include loads that are applied eccentrically and impart a twisting force to the foundation, loads that cause an unsymmetrical load to the dam pier, and concentrated loads due to tie-in details to the dam pier. These loads must be accounted for in the design, but they are normally considered an unusual condition.

b. Construction equipment. Loads due to construction equipment are normally small in comparison to other loads but should be accounted for when they are expected to be present during construction. The loads may result from both moving and stationary equipment.

c. Loads on partially cured concrete. Loads should not be placed on concrete until the concrete has achieved sufficient strength. Intermediate strength requirements should consider proposed loads, including forming and shoring systems, and concrete strength data.

d. Surcharge. In some instances, the design of vertical walls below grade will be affected by wheel loads or other surcharge loads on the ground surface. These loads should be considered in the structural stability calculations and in the detailed design as appropriate. They should be based on the heaviest piece of equipment likely to be placed on the fill during construction.

e. Seepage forces resulting from adjacent dewatering. When the newly completed or existing structure is used as a portion of the dewatering cofferdam, the adjacent dewatering causes seepage forces to act on these structures and the foundation materials below the structures. Research is currently underway at WES to evaluate the magnitude of these forces at the Mel Price Locks and Dam project.

f. Construction shoring. Shoring, or false work, is considered to be any temporary structure which supports structural elements of concrete or other material during their construction or erection. The loads due to this false work and the supported elements must be accounted for in the dam design.

9-3. Lateral Earth Loads

Lateral earth loads can be due to backfill or silt deposition. Careful investigation of available backfill materials and methods of backfilling, as well as the potential for silt accumulation, is of primary importance.

a. Backfill loads. The dam design should include the forces due to dry, naturally drained, saturated, and submerged soil conditions corresponding to the applicable loading condition, i.e., construction operation, maintenance, and so forth. The corresponding angle of internal friction of the proposed backfill material for the conditions expected to result from the proposed field placement method should also be included. Generally, "at rest" pressures should be used for gravity sections on rock and piling foundations. Values for these pressures should be determined for the various conditions of the backfill (drained, saturated, or submerged) by soil analysis methods. The lateral pressure coefficient K_o varies from about 0.3 for loose, granular soil to perhaps 1.0 for compacted clay and fine grained material. The lateral earth pressure coefficient should be determined through consultation with the geotechnical engineer. Factors that affect the lateral earth pressure coefficient are wall and foundation flexibility. See EM 1110-2-2502.

b. Silt and sediment. Horizontal pressures produced by possible silt deposits must be considered in the dam design. Model studies can only indicate tendencies for locations of silt accumulations. If the model studies indicate tendencies for silt build-up, conservative assumptions should be used. Caution should be exercised in assuming that silt and sediment will be removed, because of the

possibility that removal may not continue throughout the life of the project.

c. Vertical shear. Vertical shear, or downdrag, is the change in the state of stress in the soil backfill as vertical loading changes. These changes occur during initial construction (backfill is consolidating) and possibly during dewatering (the dewatered monolith tends to move up with respect to the stationary backfill). The procedure for computing the vertical shear is to use a vertical shear coefficient K_v applied to the effective vertical stress of the soil. The vertical shear is then applied at a plane extended at the outermost extremity of the wall. A detailed description of the procedure for gravity walls founded on rock is included in EM 1110-2-2602.

9-4. Hydrostatic

a. Horizontal water pressure. The horizontal water pressure against the dam is variable and depends on the waterway stages that prevail at a particular time or on other conditions which may produce higher pressures. For most monoliths that are not required to resist lateral earth pressures in conjunction with water pressures, the maximum pressures are easily determined. Dam abutment monoliths often have backfill adjacent to one side.

(1) No definite rule can be followed in determining the level of the groundwater in the backfill adjacent to the dam. The saturation level varies between upper and lower pool elevations, depending on the physical characteristics of the backfill material. The location of the saturation line should be based on thorough laboratory tests of the dry and wet characteristics of the soil, the extent of compaction expected, and the effect of local climatic conditions.

(2) A majority of the navigation dams in the United States are located in natural waterways where the backfill material used has granular characteristics. This material has a tendency to drain and become saturated with an approximately straight-line variation between pool elevations. For projects with fairly stable levels, these assumptions should be sufficiently accurate to give satisfactory results. However, varying pool levels and use of impervious backfill material will probably cause considerable departure from straight-line variation.

(3) For dam installations with a lower pool subject to greater fluctuations than the upper pool, a lower pool stage exceeded no more than a small part of the time should be selected from the stage duration curves. In this case, the saturation line can be constructed between this lower pool level and the normal upper pool level, and the height of the ground water table can be determined accordingly for that portion of the dam under consideration. The extent of saturated earth must be established with a reasonable degree of accuracy in order to accurately represent horizontal force due to earth, water, and uplift pressures.

(4) In addition to the usual stabilized groundwater levels caused by normal discharges, extreme loading conditions due to raised saturation levels must also be investigated. These conditions include the effect of locally heavy rains without an accompanying rise in the pool stages and of flood discharges which cause the earth to become saturated. Following flood discharges, the pool levels often approach their normal levels more rapidly than the fill material can drain. Although these increased loads are serious and should be investigated, they are normally of short duration and infrequent occurrence, and the stability requirements are usually relaxed for these increased loads. For a free-draining backfill in this condition, consider the backfill drainage as partially effective and assume, for design purposes, the saturation line to be halfway between its normal location and the top of the fill. The uplift pressure adjacent to the backfill should correspond to the assumed saturation level.

b. Uplift. Uplift due to hydrostatic pressure at the junction between the dam and foundation must be considered. Effective downstream drainage will generally limit the uplift at the toe of the dam to tailwater pressure. The uplift pressure at any point under the structure will be dependent on the presence, location, and effectiveness of foundation drains. Any existing artesian pressures should also be considered. Determination of uplift can be made using the guidance in EM 1110-2-2200. Since uplift may have a relieving effect on foundation loads, the stability of dam monoliths should be investigated for both the maximum and minimum probable uplift pressures. Dewatering systems used for construction can significantly reduce and possibly eliminate uplift and should be thoroughly investigated. A free body diagram of uplift pressures acting on the base of a dam is shown on Plate 18.

(1) Flow net and creep theory. The fundamental design principles and guidance concerning seepage considerations are detailed in EM 1110-2-1901. Additional theory is given in the documentation for the CSLIDE computer program.

(2) Geotechnical investigations. The permeability of the foundation soils greatly affects the uplift pressure;

therefore, close coordination with geotechnical engineers is needed in determining uplift pressures.

c. Vertical water. The downward force of water above the overflow section, stilling basin, flip or roller bucket, and apron should be considered to take the shape of the hydraulic profile. The shape should include the hydraulic jump of the water flowing over the section. Variations in the shape of the hydraulic profile should be investigated based on the loading condition being considered. Any contained water should also be included as a force acting downward.

d. Pulsating pressures. Pulsating pressures against sidewalls and stilling basin slabs are known to exist. The location and magnitude of these pressures can be investigated with a hydraulic model as was done for Baldhill Dam (Fletcher 1993). EM 1110-2-2400 recommends increasing the static load by 1.5 to account for pulsating pressures. These pressures should be accounted for in the design of stilling basin slabs and sidewalls.

9-5. Earthquake or Seismic

Two general approaches to determining seismic forces include the seismic coefficient method and a dynamic analysis procedure.

a. Seismic coefficient method. The seismic coefficient method (also known as the pseudo-static method) of analysis should be used only as a preliminary means of determining the location of resultant and sliding stability of monoliths, or to provide an initial pile layout. If the seismic loads computed by the seismic coefficient method indicate a critical load case, a more rigorous dynamic analysis should be performed as described in Chapter 11. The coefficients used are considered to be the same for the foundation and are uniform for the total height of the monolith wall. Seismic coefficients used in design are based on the seismic zones provided in ER 1110-2-1806. Details of these procedures are contained in EM 1110-2-2200.

b. Dynamic analysis procedure. Procedures for performing a dynamic analysis are contained in ETL 1110-2-365.

9-6. Tow Impact

Tows operating on inland waterways on occasion lose control and collide with dam piers and appurtenant structures. (This load is not applied concurrently with ice load.) The magnitude of the impact forces generated by a particular collision depends on numerous factors such as size, speed, and angle of tow; stiffness of object being struck; and stiffness of barge. An analytical approach which can be used to approximate the maximum impact forces on structures located on the inland waterway system is presented in ETL 1110-2-338.

9-7. Line Loads

When check posts or line hooks are provided on dam structures, a hawser pull of 160 kips should be used for the design of the posts or hooks and their anchorages.

9-8. Ice and Debris

The magnitude of the ice load to be figured into the design of dams should be estimated for the particular structural element being designed, with consideration given to locale and available records of ice conditions. The effect of wedging ice flows between piers should be considered, and the most unfavorable direction of ice load chosen. Accepted practice has been to assume a load of 5,000 lb/ft of width of dam to account for impact of debris and ice loads. For more detailed methods of computing ice forces, see EM 1110-2-1612.

9-9. Wave Loads

Wave loads are usually more important in their effect on gates and appurtenances, but they may in some instances have an appreciable effect on the design of the dam structure. Wave loads are not calculated concurrently with the ice and debris loads. Wave dimensions and forces depend on the extent of water surface or fetch, the wind velocity and duration, and other factors. More information relating to waves and wave pressures is presented in CERC's "Shore Protection Manual" (SPM), Volume 111 (SPM 1984).

9-10. Wind Loads

Wind and subatmospheric pressures may ordinarily be neglected in analyses for low-navigation dams. Wind pressures on the exposed piers, service bridge, crane, etc., should be assumed to act in the most unfavorable direction, and also should be assumed as 30 lb/sq ft (corresponding to a wind velocity of 85 mph). The load should be assumed to act on the following surfaces:

a. Bridge girders - One and one-half times the vertical projection of the span.

b. Bridge trusses - The vertical projection of the span plus any portion of the leeward trusses not shielded by the floor system.

c. Crane - The vertical projection of the crane.

d. Pier - The vertical projection of the exposed parts of the pier and accessories.

For locations where the wind velocities exceed the previously stated velocity, correspondingly higher wind load allowances should be used. TM 5-809-1 contains additional guidance on wind loading.

9-11. Gate Loads

All reactions determined from analysis of the gates should be accounted for in the design of the dam. These include anchorage forces from tainter gates, wickets, hinged-crest gates, and other gates. Internal forces from post-tensioned tainter gate trunnion girders must also be considered in the design of dam piers. Gate piers are subject to eccentric lateral loading, such as that resulting from the raising of one gate while the adjacent gate remains closed. The monolith stability and the pier design should reflect these loading conditions. The torsional shear from this loading should be considered, and the pier should be reinforced appropriately.

9-12. Bridge Loads

The bridge should be designed for dead and live loads including crane and bulkhead loads, as well as wind and seismic loads. Bridge loads are imparted to the dam structure as a result of piers that support a service, access, pedestrian, or highway bridge. Bridge loads should be considered in the stability computations as well as in determining localized stresses in the dam pier.

9-13. Crane Loads

Loads due to cranes and other machinery can be significant and must be included in the analysis of the structure. The loads may result from both moving and stationary equipment, such as bulkhead handling equipment and tainter gate hoists. These loads should be applied as point loads at the appropriate locations.

9-14. Bulkhead Loads

Reactions from maintenance or emergency bulkheads should be accounted for in the design of the dam. Gate piers are subject to eccentric lateral loading, such as that

resulting from the dewatering of one gatebay while the adjacent gatebay remains operational. The torsional shear from this loading should be considered, and the pier should be reinforced appropriately.

9-15. Sheet Pile Cutoff Loads

A sheet pile cutoff wall can impart a load to the structure if the sheet pile wall is subject to an unbalanced loading, as in the case where uplift pressures on each side of the sheet pile wall vary due to a pressure relief system or seepage losses. Past analyses showed this load to be negligible in comparison to other loads except in the case of sheet pile driven deep with a high section modulus. In this case, the load should be based on an analysis similar to that presented in EM 1110-2-2602, Chapter 8.

9-16. Monolith Joint Loads

a. Waterstop related. Pressures in monolith joints should be evaluated based on the critical condition of waterstops being either ruptured or intact to give the worst case.

b. Keying between structures. Dam structures are typically designed so each monolith acts as a separate structural unit independent of adjacent monoliths. However, in some cases it may be beneficial to key monoliths together. A typical case is where the upstream end of a stilling basin is keyed to the downstream end of a dam monolith to satisfy stability requirements for the stilling basin. Both monoliths should be designed to withstand all forces that may be transferred across the joint for both stability and local considerations. A typical keyed joint is shown in Plate 2. See paragraph 11-3*f* for additional information on dowels and reinforcing between joints.

c. Reinforcing bars. Dowels or reinforcing bars can be used to prevent differential movement between monoliths, in both the lateral and the longitudinal direction. The reinforcing must be designed to account for the complete load transfer between monoliths, and the monoliths designed to withstand all forces that may be transferred across the joint for both stability and local considerations.

9-17. Superstructure Loads

Superstructure loads are imparted to the dam structure as a result of appurtenant structures such as machinery rooms placed upon the top of the dam piers. Loads from these structures should be considered in the stability computations as well as in determining localized stresses in the dam pier.

9-18. Thermal Loads

Thermal loads may be significant in determining cracking potential for a concrete monolith and also in determining residual stresses which would be additive to seismic loads. This topic is covered in Chapter 12.

Chapter 10
Design Criteria

10-1. Applicability and Deviations

The design criteria set forth in this chapter apply in a general sense to the design and analysis of dam structures. Conditions that are site-specific may necessitate variations which must be substantiated by study and testing of both the structure and the foundation.

10-2. Load Cases

a. Usual. Dam structures are designed for usual load conditions, those that occur most commonly during the life of a project, including both normal operating and frequent flood conditions. Basic allowable stresses and safety factors apply in these cases.

b. Unusual. Higher allowable stresses and lower safety factors may be used in accounting for unusual loading conditions such as maintenance, infrequent floods, barge impact, construction, hurricanes, or earthquakes with nonspecific ground motions for OBE (operating basis earthquake). For these conditions, allowable stresses may be increased up to 33 percent. Lower safety factors for pile or foundation capacity may also be used.

c. Extreme. High allowable stresses and low safety factors are used for extreme loading conditions such as accidental or natural disasters that have a remote probability of occurrence and that involve emergency maintenance conditions such as earthquakes with nonsite-specific ground motion for MCE (maximum credible earthquakes). For these conditions, allowable stresses may be increased up to 75 percent. Low safety factors for pile or foundation capacity may be used as described for unusual loads. Special provisions (such as field instrumentation, frequent or continuous field monitoring of performance, engineering studies and analyses, and constraints on operational or rehabilitation activities) are required to prevent catastrophic structure failure during or after extreme loading conditions. Deviations from these criteria for extreme loading conditions should be formulated in consultation with and approved by CECW-ED.

10-3. Earth and Rock Foundations

Generally, an earth- or rock-founded structure is the most cost-effective foundation alternative. A prime consideration in selecting a foundation system is differential settlement. Deflection and differential settlements must be within acceptable limits for the serviceability of the gates and other operating equipment, and adequate stability must be provided. Adequate stability is attained by specific limitations on the magnitude of the foundation pressure (bearing capacity) and the resistance to sliding, and on the location of the resultant of the applied forces within the base of the structure.

a. Foundation pressure.

(1) In general, allowable foundation pressures should not be exceeded for any loading condition; however, the allowable values may be different for usual and extreme load cases. For comparison, only one allowable foundation pressure per material should be used, and an increase of one-third should be allowed for unusual and extreme load case categories. For bearing capacity, EM 1110-1-1905 allows a safety factor of 2.0; however, current practice in the Corps is to use 3.0 for usual load cases and 2.0 for unusual or extreme load cases.

(2) Base pressure computation should be made by uniformly distributing the normal component of the resultant of all forces on the structure (including uplift) as a reaction on the base (or plane of investigation) by means of the general flexure formula or by equations of equilibrium. Uplift should be adjusted in areas of non-compression. Foundation pressure is equal to base pressure plus uplift pressure. Therefore, the stability design should be checked using full uplift forces for overturning and without uplift forces for maximum foundation pressures.

b. Sliding.

(1) Purpose. The purpose of a sliding stability analysis is to assess the safety of a structure against a potential failure due to horizontal movement. The potential for sliding failure may be assessed by comparing the applied shear forces to the available resisting shear forces along an assumed failure surface. A sliding failure is imminent when the ratio of the applied shear forces to the available resisting shear forces is equal to one.

(2) Soil rock shear strength. The shear strength of the soil and/or rock that comprises the foundation (failure surface) is sensitive to the duration of the load, the soil's ability to drain, the saturation elevation, the number of layers, and many other conditions. Due to these sensitivities, a fully coordinated team of structural, hydraulic, and geotechnical engineers and geologists should be formed to ensure that all pertinent engineering considerations are adequately integrated into the analysis and the correct shear strengths are used.

(3) Analysis model.

(a) The shape of the failure surface may be irregular, depending on the homogeneity of the backfill and the foundation material. The failure surface may be composed of any combination of plane and curved surfaces. However, for simplicity, all failure surfaces are assumed to be planes which form the bases of wedges, as shown in Figure 10-1.

(b) Except for very simple cases, most sliding stability problems encountered in engineering practice are statically indeterminate. To reduce an indeterminate problem to a statically determinate one, the problem must be simplified by dividing the system into a number of rigid body wedges. This division arbitrarily assumes the direction of the equilibrium forces which act between the wedges and neglects any frictional forces between adjacent wedges.

(c) The failure surface can be divided into wedges, as shown in Figure 10-1. In this example, the base of a wedge is formed from a section of the failure surface that lies in a single soil material or along the base of the structure. The interface between any two adjacent wedges is assumed to be a vertical plane which extends from the intersection of the corners of the two adjacent wedges upward to the top soil surface. The base of a wedge, the vertical interface on each side of the wedge, and the top soil surface between the vertical interfaces define the boundaries of an individual wedge.

(d) In the sliding analysis, the dam monolith and the surrounding soil are assumed to act as a system of wedges, as shown in Figure 10-1. The soil-structure system is divided into one or more driving wedges, one structural wedge, and one or more resisting wedges.

(e) Depending on the geologic conditions of the foundation material, the location of the total failure surface or parts of the failure surface may be predetermined. Natural constraints at the site may also predetermine the inclination of some of the failure planes or the starting elevation of the failure planes adjacent to the structure. Conditions which warrant the predetermination of parts of the failure surface include bedding planes and cracks in a rock foundation.

(4) Analysis procedure of the soil-structure system. An iterative procedure can be used to find the critical failure surface. For an assumed factor of safety (FS), the inclination of the base of each wedge is varied to produce a maximum driving force for a driving wedge or a minimum resisting force for a resisting wedge. The assumed FS is varied until a failure surface is produced that satisfies equilibrium. The failure surface which results from this procedure will be the one with the lowest FS. Finite element analysis procedures may also be used.

(5) Sliding factor of safety (FS). Limit equilibrium analysis is used to assess stability against sliding. An FS is applied to the factors which affect the sliding stability and are known with the least degree of certainty. These factors are the material strength properties. An FS is applied to the material strength properties in a manner that places the forces acting on the structure and soil wedges in equilibrium. Because the in-situ strength parameters of rock and soil are never known exactly, one role of the FS is to compensate for the uncertainty that exists in assigning single values to these important parameters. In other words, the FS compensates for the difference between what may be the real shear strength and the shear strength assumed for the analysis. Sliding stability criteria for navigation dams are listed below:

- Usual   2.0

- Unusual   1.7

- Extreme   1.3

(6) Detailed design. Detailed design procedures and multiple wedge derivations can be found in ETL 1110-2-256.

(7) Computer programs. The computer program CSLIDE can assist in performing a multiple wedge sliding analysis.

c. Location of resultant. The location of the resultant of all forces within the base (or limits of the plane of investigation) determines what percentage of the base is in compression. See Figure 10-2. If the resultant lies within the kern (middle third), then the entire base is in compression. This requirement applies to usual load cases. If the resultant lies outside the kern but within the base, then only a portion of the base area is in compression. This portion can be expressed as a percentage of the base computed from the general flexure formula or from the equilibrium equations. At least 75 percent of the base should be in compression for unusual load cases. This measurement is consistent with an eccentricity not exceeding one-fourth of the base length, and it is a suitable approximation for all base shapes. Because the resultant
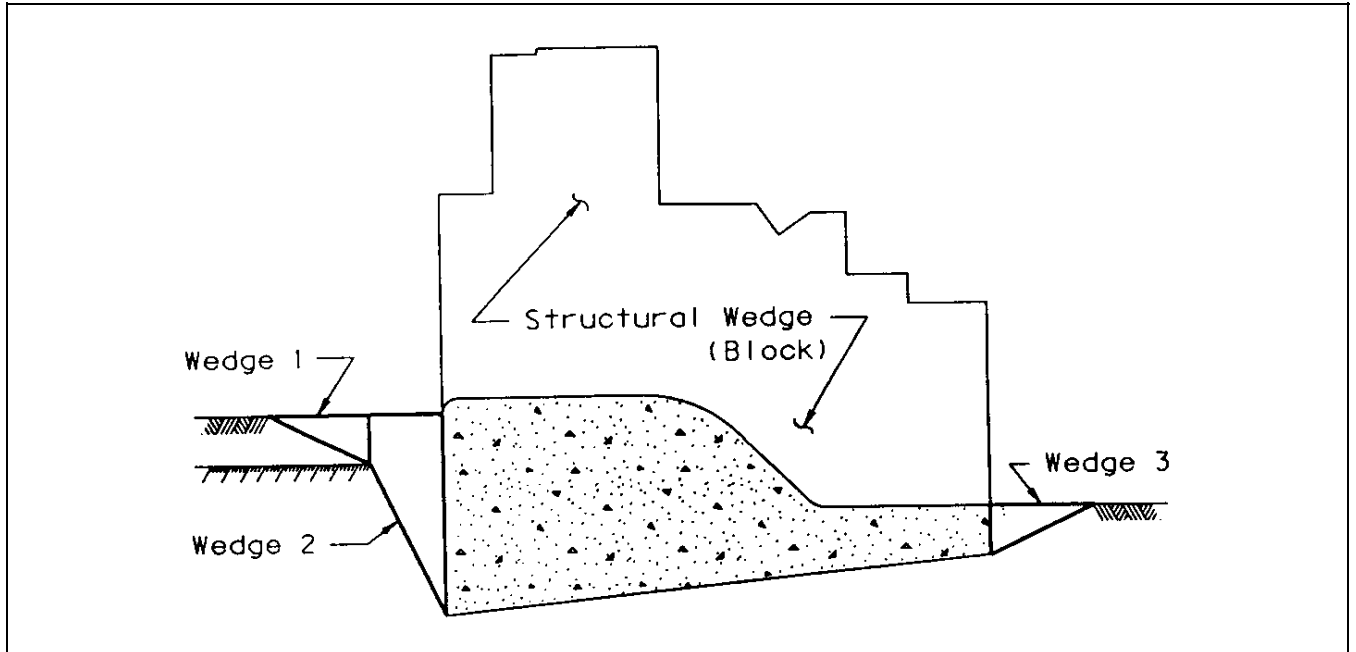
Figure 10-1. Wedge analysis model
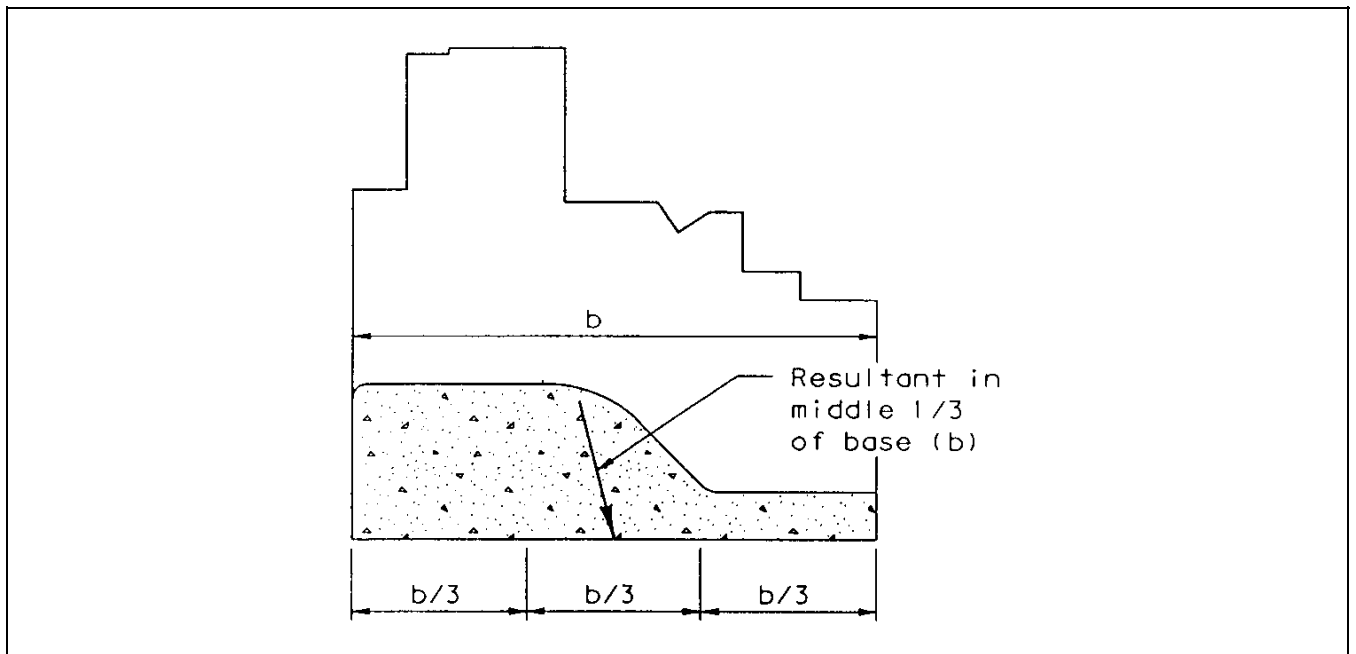
Figure 10-2. Location of resultant

is required to fall within the base only for extreme load cases, the corresponding percentage of the base in compression will be "greater than zero." These specific requirements are true only for structures of unit width or with rectangular bases and subject to bending about one principal axis; however, these concepts are extended to three-dimensional (3-D) structures with irregular bases and subjected to biaxial bending loads.

d. Settlement. Foundation pressures should not produce total differential settlements that result in operational difficulties (e.g., improper operation of gates and rupture of water stops). In locations where detrimental settlement of dam foundations might occur, a settlement analysis should be made as presented in EM 1110-1-1904. If the settlement analysis indicates a possible concern, the settlement should be corrected by extending the foundation to reduce base pressures, designing alternative foundation (piles), using an alternative site, or using means to ensure that monoliths move together as discussed in Chapter 9.

10-4. Internal Stability

For gravity sections, structural adequacy within the body of a section is attained by limiting internal stresses to values which do not exceed the safe working stresses of the material under stress. Internal instability will be a result of overturning forces and excess pore pressure. Pore pressures may be estimated by methods presented in EM 1110-2-2602. In general, horizontal planes above the foundation are required to have the resultant force inside the kern. For usual load cases, this requirement limits

stresses normal to the plane to the compressive range only. For unusual load cases, 75 percent of the base should be in compression when the normal stress component does not exceed the permissible tensile stress for the material (e.g., maximum fiber stress in plain concrete due to factored loads and moments shall not exceed a tensile stress of $0.05f'_c$).

10-5. Uplift and Flotation

These items are closely related in meaning, and both usually act to minimize the degree of structural stability with respect to sliding and overturning. Uplift is determined from seepage analysis by methods presented in Chapter 9. The stability analysis with regard to flotation should be done in accordance with EM 1110-2-2602.

10-6. Pile Criteria

If deflection and differential settlements are not within acceptable limits for the serviceability of the gates and other operating equipment, and adequate stability cannot be attained, a pile foundation should be considered. The pile cap (dam structure) should be modeled as either a rigid block or a flexible base consistent with flexural properties of the pile cap. The pile response is usually based on linear elastic behavior with limiting axial and lateral deflections of 1/4 and 1/2 in., respectively. In general, the FS for axial pile capacity varies between 1.15 and 3, depending on the method of predicting the capacity and the loading condition. Detailed guidance on pile foundation design is provided in EM 1110-2-2906.

Chapter 11
Analysis and Design

11-1. General

This chapter addresses the criteria, procedures, and parameters necessary for the analysis and design of the foundation system and the dam structure. Design of the project involves in-depth study of soil/structure interaction, stability analysis, and structural analysis.

11-2. Structural Analysis

a. General. A dam typically consists of a series of 3-D concrete structures (monoliths) surrounded by soil/silt, rock, and water. These structures are founded on either rock, soil, or piles and are subjected to a variety of external and internal soil and water loads.

b. Two-dimensional (2-D) analysis. An analysis of a 2-D slice through a monolith can reliably indicate the behavior of the monolith under the following conditions:

(1) When the cross-section geometry of the structure, the soil and water conditions, the support conditions, and the other loading effects are constant throughout an extended length of the monolith.

(2) When a 2-D slice, obtained by passing parallel planes perpendicular to the longitudinal axis of the monolith, typifies adjacent slices and is sufficiently remote from any discontinuities in geometry and loading (i.e., the slice is in a state of plane strain).

c. Two-dimensional (2-D) frame analysis. Structural analysis of the dam component is based on the assumption that the various slabs, walls, etc., of the structure interact as elements (members) of a 2-D plane frame. Establishment of a plane frame representation of the structure requires designation of parts of the structure as flexible members connected at their ends to joints. While some regions of the structure may lend themselves to treatment as flexible members (i.e., beam bending elements), there are significant zones of mass concrete that cannot be assigned bending characteristics. For analysis purposes, these zones are assumed to be rigid. The location and extent of these rigid zones will depend on the type of monolith being analyzed. The size of these zones must be determined to obtain reliable indications of the behavior of the 3-D monolith using a frame model. The frame should be calibrated by determining rigid zone sizes to agree with the stress results from a finite element analysis using plane strain elements for a section with similar geometry.

d. Three-dimensional (3-D) analysis. If the dam monolith geometry and/or loading does not meet the above requirements for a 2-D frame analysis, a 3-D finite element computer model should be used to analyze the monolith. Guidance on modeling of structure for linear elastic finite element analysis is provided in other Corps documents.

e. Seismic. Earthquake-induced ground motion effects must be considered in the analysis and design of navigation dam structures. The structures must be designed for the inertial forces from the structure mass combined with hydrodynamic pressures. These forces should be combined with any dynamic soil pressures generated within the backfill. Linearly elastic procedures used in design include the response spectrum analysis and the time history analysis.

(1) Seismic coefficient method. Traditional design practice based on the seismic coefficient method failed to account for the dynamic response characteristics of the soil-structure-water system. Dams designed by the seismic coefficient methods may not be adequately proportioned or reinforced to resist forces generated during a major earthquake. Therefore, this approach should be used only as a simple, preliminary means of checking a new design or an existing structure for seismic susceptibility. It should not be used as a final analysis procedure for controlling member proportions or for remedial design (with the exception of those cases where extensive results or comparisons of previously designed or evaluated structures are available).

(2) Response spectrum analysis. A response spectrum is a plot of the maximum response of a series of single-degree-of-freedom (SDOF) systems with varying periods or frequencies. A response spectrum analysis can provide an analysis procedure that partially accounts for the dynamic structural properties of the system. The response spectrum analysis can be accomplished by either a finite element procedure or a frame analysis. Results from these procedures provide only the *absolute* maximum stresses and forces due to the methods of combining modal responses.

(3) Time history analysis. The exact time history of a response quantity can be produced using this technique; therefore, an exact sign-dependent stress distribution can be found at any given time. However, a digitized design earthquake record for the site is needed, and a significant

computing effort is required for the numerical integration of the differential equation of motion using small time steps.

(4) Guidance. A detailed description of the response spectrum and the time history analysis is provided in ETL 1110-2-365.

f. Nonlinear incremental structural analysis (NISA). A NISA should be conducted on massive concrete structures if it will help achieve cost savings, develop more reliable designs for structures that have exhibited unsatisfactory behavior in the past, or predict behavior in structures for which a precedent has not been set. A NISA first requires that a time-dependent heat transfer analysis be performed. Further discussion on NISA is included in Chapter 12.

11-3. Foundation Design and Soil/Structure Interaction

a. Type of foundation. Another critical aspect in the design of navigation dams involves determining the appropriate foundation type. The foundation conditions often influence the site selection for a navigation lock project. The foundation characteristics should therefore be determined for each tentative site at an early stage of the investigation. These characteristics are usually determined by using available data and a minimum of foundation exploration. Sites chosen for further investigation should have foundation characteristics that would allow the dam structures to be constructed at a reasonable cost. The possible sites selected for study from a review of topography and hydraulics can thus be reduced to one or two after reviewing the site from a foundation and navigation standpoint. Final site selection requires extensive foundation exploration of the remaining sites under consideration. Before a pile foundation is selected, the foundation characteristics must be well-defined and a sufficient analysis of them must be made.

b. Foundation pressures (compatible deformations). Foundation pressures depend on the type of foundation material, the nature of the loading, and the size and shape of the monolith. For gravity-type monoliths (due to their rigidity), a linear distribution of base pressure can be assumed. However, for structural monoliths with a flexible base, the distribution of base pressure should be based on a soil/structure interaction analysis.

c. Bearing strength of soils. The bearing strength of soils and methods for its determination based on field and laboratory test data are described in EM 1110-1-1905.

Another good reference for the calculation of bearing capacities is the program documentation for the CASE computer program CBEAR.

d. Earth pressures settlement. For a gravity structure, settlement analyses can be performed by following the principles set forth in EM 1110-1-1904.

e. Pile foundations.

(1) Determination of type of foundation--soil or pile.

(a) Determining the foundation type is probably the most critical aspect in the design of a dam. Because this decision will affect the project cost, the foundation type should be determined at the feasibility report stage of the project. This analysis should involve the use of a thorough subsurface investigation and testing program to define the soil strengths and parameters. For major structures an in-situ pile load test will normally be required.

(b) The criteria for selecting a soil or pile foundation are based on economic considerations and site-specific characteristics. Usually, a soil foundation is more economical if special measures (deeper excavation, elaborate pressure relief system, etc.) are not required. In addition, the structure on a soil foundation must satisfy stability requirements for sliding and overturning, as well as resisting uplift (flotation) and earthquake forces. At some sites, liquefaction of the foundation in the event of an earthquake becomes a determining factor in selecting the foundation type. Differential settlements between monoliths should also be considered in determining whether a soil or pile foundation will be used. If expensive special measures are required to make a soil foundation suitable for use, then a pile foundation should be studied and its cost compared to the cost of a soil foundation. The piling selection process should consider all reasonable types of piling, the site's geotechnical conditions, availability of material, construction limitations, and economics. The estimated quantities of piling can be based on minimum spacing and approximate lateral and vertical capacities for one or two typical monoliths. The most cost-effective type of pile foundation that satisfies engineering requirements is thus determined for comparison to the soil foundation. Computer programs such as CPGA (rigid base) or CWFRAME (flexible base) or other finite-element programs are useful for designing pile foundations. The final decision between a soil and a pile foundation is then based on a cost comparison using these refined pile quantities.

(2) Design guidance. Detailed design guidance for pile foundations is contained in EM 1110-2-2906.

f. Design considerations. Dowels or reinforcing bars can be used to prevent differential movement between monoliths. These may be bonded, bonded on one side, or greased on each side of the joint, and they may be post-tensioned. The reinforcing should be detailed to facilitate construction and so that it is not exposed to water in the joint. See paragraph 9-16*b* for a discussion of joint loads.

Chapter 12
Concrete Design

12-1. General

Most navigation dams are constructed as massive concrete structures (defined by American Concrete Institute Committee 207 as "any large volume of cast-in-place concrete with dimensions large enough to require that measures be taken to cope with the generation of heat and attendant volume changes to minimize cracking"). See EM 1110-2-2000 for a complete discussion of standard practice for concrete construction. There are three types of massive concrete structures commonly used for civil works projects: gravity structures, thick shell structures, and thick reinforced plates. Selection of concrete which meets design requirements is critical to long-term performance of these structures. The design requirements generally include the following characteristics:

a. Strength. Strength is an important characteristic of concrete. Typically, 3000-psi- or 4000-psi-strength concrete is utilized, but lower interior strengths or higher strengths may be used as well. Concrete strength may be varied by zones and may not always be limited to the normal 28-day designation. A 90- or 120-day strength, which is characteristic of concrete with a high percentage of pozzolan substitution for cement, may be utilized for long-term construction projects.

b. Durability. Concrete must resist deterioration by the environment to which it is exposed, including freezing and thawing, wetting and drying, chemicals, and abrasion.

c. Placeability. Placeability is described by the terms workability and consistency. These are affected by many factors, including water content, cement or pozzolan, and maximum size aggregate--the last of which, in turn, is influenced by the presence of steel reinforcement or embedded items.

d. Economy. The maximum economy can be achieved by minimizing the amount of cement utilized and, where appropriate, replacing portland cement with generally less expensive pozzolans. Economy is also improved by using the maximum size aggregate consistent with the dimensional requirements of the structures on the project, and by using aggregates available to the project.

e. Ductility. In areas of high seismic ground motions, the tensile strain capacity may be of great importance.

12-2. Nonlinear Incremental Structural Analyses

Determining the appropriate measures to cope with the problems associated with a massive concrete structure, including the division of the structure into separate elements such as piers, sills, or monoliths, is an important task. For major structures, a NISA should be used as a design for massive concrete structures if it will help achieve cost savings, develop more reliable designs for structures that have exhibited unsatisfactory behavior in the past, or predict behavior in structures for which a precedent has not been set. A NISA requires that a time-dependent heat transfer analysis be performed. The results of the heat transfer analysis are then used in a time-dependent stress analysis that simulates the incremental construction of the structure and uses nonlinear properties for modulus of elasticity, creep, and shrinkage. For more information on performing a NISA, refer to ETL 1110-2-324.

12-3. Parameters Affecting Cracking in Concrete

a. General. Cracking in non-massive reinforced concrete structures is primarily the result of tensile strains produced by loads applied to the structure. Steel reinforcement is provided to carry the tensile stresses. Cracking in mass concrete is primarily caused by restraint of volume change due to heat generation and subsequent cooling, autogenous shrinkage, creep/stress relaxation, or other mechanisms. Restraint limits the respective changes in dimensions and causes corresponding tensile, compressive, torsional, or flexural strains in the concrete. Of primary concern in mass concrete structures is restraint which causes tensile stresses and corresponding tensile strains. Restraint may be either external or internal. External restraint is caused by bond or frictional forces between the concrete and the foundation or underlying and adjacent lifts. The degree of external restraint depends on the relative stiffness and strength of the newly placed concrete and the restraining material and on the geometry of the section. Abrupt dimensional changes or openings in a monolith, such as wall offsets, gallery entrances and offsets, and reentrant corners, have caused external restraint that has resulted in cracking in concrete structures. Internal restraint is caused by temperature gradients within the concrete. The warmer concrete in the interior of the mass provides restraint as the concrete in the periphery of the mass cools at a different rate due to heat transfer to its surroundings. The degree of internal restraint depends upon the total quantity of heat generated, the severity of the thermal gradient, the thermal properties of the concrete, and thermal boundary conditions.

b. Geometry. The geometry of the structure is of course a major contributing factor to the behavior of the structure. Therefore, a NISA should not be performed until the structural geometry is at a stage where only minor changes to it are expected. While this parameter may be more difficult to alter than others, there may be instances in which a change is necessary. If such a change is made to the geometry of the structure, then coordination between all disciplines is necessary to ensure the change does not have an adverse effect on some other function of the structure. A change in the geometry will generally require some type of revision to the model's mesh.

c. Reinforcing. Reinforcing is an integral part of non-massive concrete structures and of many of the massive concrete structures used within the Corps of Engineers. Reinforcement for all non-massive concrete structures should be designed in accordance with EM 1110-2-2104. See EM 1110-2-2000 for a comprehensive discussion of standard practice for concrete construction. For large wall and floor sections, reinforcement spacing should generally be set at 12 in. for ease of construction. In non-massive concrete sections, temperature and shrinkage reinforcement is required to control cracking. Generally, small bars at close spacing provide the best control. However, for walls 2 ft thick or more, number 9 bars at 12-in. spacings are commonly used to ease construction while still providing the required steel percentage. In gravity walls, however, the requirements of EM 1110-2-2104 regarding the minimum steel do not apply. To date there has been limited use or recognition of reinforcing in NISA analyses, due to the fact that many of the structures analyzed had no cracking problems, and adding reinforcing in the model when cracking is not occurring has little effect on results. If an analysis predicts cracking in a structure and measures to eliminate the cracking are unsuccessful, then reinforcement should be included in the model. Resulting stresses in the reinforcing bars should be monitored, reported, and compared to the yield strength of the reinforcing. If cracking is occurring in a location of minimal reinforcing or at corners of openings, increasing the amount of steel transverse to the crack can help control or arrest the crack. Typically, reinforcing steel placed at 45 deg at corners is very effective at arresting corner cracking. Special attention must be paid to providing proper concrete cover for all reinforcing bars (see EM 1110-2-2104).

d. Seismic. Reinforcing steel must be properly contained to ensure good performance during an earthquake. See ETL 1110-2-365 for further guidance.

e. Material parameters.

(1) A number of material parameters can be controlled to limit cracking related to restrained volume change. They include heat generation of the concrete; mechanical properties of the concrete, including compressive and tensile strength, tensile strain capacity, modulus of elasticity, linear coefficient of thermal expansion, creep/stress relaxation, and autogenous shrinkage; and thermal properties of the concrete, including specific heat and thermal conductivity.

(2) These properties are governed by the selection of materials used to make the concrete, including cementitious materials (portland cement, ground granulated iron, blast furnace slag, and pozzolans such as fly ash), aggregates, chemical admixtures, etc.; and by the proportions of these materials in the concrete mixture. Many of these properties are also dependent on the maturity of the concrete and are thus time and temperature dependent. Close scrutiny of the selection of concrete mixture materials and proportions should be part of a properly conducted concrete materials study. Due consideration should be given to the performance and economy of the selected mixture. The study should be conducted according to the guidance in EM 1110-2-2000 and documented in a concrete materials design memorandum.

f. Construction procedures. A number of construction parameters can be controlled to limit cracking due to restrained volume change. They include lift height, time between placement of lifts, concrete placement temperature, curing method, use of insulation, monolith geometry including section thickness, monolith length, and location and size of inclusions such as galleries. In addition, the time of year a monolith is constructed can be controlled if it has been determined by the NISA that a particular start date is beneficial. Any construction requirements or restrictions identified by the NISA must be clearly stated in the construction contract documents.

g. Vertical construction joints within a monolith. There may be some projects for which vertical construction joints become necessary due to excessively large concrete placements. If this is the case, lift sequences creating vertical joints should be accounted for in the incremental construction analysis procedure. Stresses across a vertical construction joint should be examined closely for determination of any special measures needed in the design and construction of the joint (e.g., placement of reinforcement bars across the joint face). In addition, a 3-D analysis should be considered for monoliths with

vertical construction joints to confirm results obtained in the 2-D analysis, because the joints themselves will be located in the out-of-plane direction.

12-4. Concrete Quality for Dam Spillway and Stilling Basin

a. Spillway function and composition. The primary function of a navigation dam concrete spillway is to provide a controlled release of surplus water from the impounded pool so that the water level can be regulated for navigation traffic use. The spillway may be composed of the following features: ogee spillway crest and shape, stilling basins, apron, bucket, end sill, and baffle blocks, as well as spray walls, separation walls, and training walls.

b. Concrete durability. The concrete for all the above features must be able to resist the effects of environmental deterioration, including freezing and thawing, wetting and drying, chemicals, and abrasion. In addition, the concrete should be able to resist damage caused by waterborne rocks and gravel, floating ice, floating tree trunks, and debris of all kinds in the high-velocity flowing water. Damage to the concrete in spillways and stilling basins is a constant maintenance problem on many existing Corps projects. On various projects, abrasion-erosion has ranged from a few inches to several feet, and, in some cases, severe damage has occurred after only a few years of operation. The fact that many spillways cannot be easily or economically unwatered for inspection and repair makes the initial quality and the placement of the concrete all the more important. To protect the spillway concrete from all the potentially damaging elements, durable concrete mixes and proper concrete placement requirements should be provided in accordance with all the recommendations outlined in EM 1110-2-2000. Special attention must also be given to providing proper concrete cover over steel reinforcement (see EM 1110-2-2104).

12-5. Second Placement Concrete

a. Purpose. Second placement concrete is necessary for a dam structure at locations where precise settings are required for alignment and/or elevations for embedded steel items. These items and locations may include horizontal seal plates on spillway crests for spillway gates, vertical side sealing and rubbing plates for "J" seals attached to spillway gates, machinery bases for support of gate operating machinery, horizontal seal plates for spillway bulkheads, vertical plates for vertical-lift gates and spillway bulkheads to bear on and roll on, corner protection for spillway bulkhead slots (upstream and downstream), dogging devices for spillway bulkheads, and crane rails for a movable crane located on the service bridge.

b. Design consideration. These second placement blockouts require careful sizing and detailing so that enough space is available for adjusting the steel items to line and grade and to allow for placing and vibrating concrete. The following items also require special attention: the concrete mix (which must be designed with the proper aggregate size to allow good placement); the concrete mix design (which must provide for minimum shrinkage); reinforcing steel extending from the mass concrete placement; additional steel for the second placement; and embedded bolts and adjustment provisions for securing the steel items in position. For typical second placement details, see Plate 19. EM 1110-2-2000 contains guidance in proportioning and constructing blockout concrete.

Chapter 13
Design of Other Items

13-1. Galleries

A system of galleries, adits, chambers, and shafts is usually provided within the body of the concrete dam to furnish access and space for foundation drilling and grouting and for the installation, operation, and maintenance of equipment accessories and utilities in the dam. The primary considerations in arranging the required openings within the dam are personnel safety, functional usefulness and efficiency, and location of the openings with respect to maintaining the structural integrity of the structure.

a. Ventilation. In accordance with EM 385-1-1, all of the above-mentioned galleries and other associated features should be provided with adequate ventilation for safe operation, maintenance, and inspection. Adits should be equipped with doors of structural steel grillages that allow free circulation of air in and out of the galleries. This arrangement may be supplemented by using vent holes in the galleries and at other selected locations and by placing powered exhaust fans in special areas. Hydrogen sulfide gas in the galleries is a hazard to personnel. Ventilation equipment where this gas occurs must be explosion-proof.

b. Required dimensions. A gallery for the grouting of the foundation cutoff will extend the full length of the dam. It will also serve as a collection main for the seepage from the foundation drainage holes. See Figure 7-1. The location of the gallery should be near the upstream face of the dam and as near to the rock surface as structural design and layout will allow. It has been standard practice to provide grouting galleries 5 ft wide by 7 ft high. Experience indicates that increasing these dimensions can facilitate drilling and grouting operations. Where practicable, the width may be increased to 6 ft and the height to 8 ft. The floor of the gallery should slope about 1/4 in. per foot to a minimum-size 12-in. by 12-in. gutter along the upstream side. The depth of the gutter will vary so that the bottom slopes for proper water flow to the collection sump. The gallery is usually arranged as a series of horizontal runs and stair flights as dictated by the varying foundation levels. The stairs should be provided with safety treads or a nonslip aggregate finish. Where it is probable that equipment will be skidded up or down the steps, metal treads are preferable since they provide protection against damage to the concrete. Where practicable, the width of tread and height of riser should be uniform throughout all flights of stairs and should never change in any one flight. Details of drilling and grouting operations and equipment are covered in EM 1110-2-3506.

13-2. Machinery Platforms

a. Reinforced concrete. A reinforced concrete platform is readily adaptable for tainter gate hoist machinery support--especially for cases where the weight of the machinery and components plus the tainter gate and lifting cables creates a very heavy loading on the platform. The heavily reinforced concrete support members are cantilevered off the top of the pier. If a service bridge or walkway bridge is provided, the machinery base level and the bridge level will be at approximately the same elevation. A heavy structural steel frame, anchored to the concrete with embedded anchor and adjusting bolts, provides support for the machinery and components. Typically, a minimum of 1-in. cement grout is placed under the steel frame after it is adjusted to the proper location and elevation. The machinery and components are then bolted to the top of the frame. See Plate 20 for layout of a typical structural steel frame for the hoist machine and components it supports.

b. Steel grillage with concrete deck or grating. On smaller navigation dams where loads are not so large, a cantilevered structural steel support assembly may be more adaptable for use than a reinforced concrete support. The structural steel assembly is anchored to the top of the pier with embedded anchor and adjusting bolts. After the support is installed to line and grade using second-pour concrete, the machinery and components are bolted to the top of the support assembly. The openings in the support assembly are sometimes covered with steel grating so that personnel will have access to all components of the hoist machinery for maintenance, inspection, and repair activities.

c. Design loadings. In addition to its own dead weight and the weight of the hoist machinery and components, the cantilevered platform must be designed for the following loads: dead weight of the tainter gate, side seal friction, dead weight of wire ropes or chains, trunnion friction, silt and drift accumulation on gate, ice accumulation, impact, and stall torque of electric motor. See EM 1110-2-2702 for further information and guidance on hoist loadings.

13-3. Machinery Houses

a. Requirements and optional usage. In locations where climates are severe and the seasonal icing, rain, wind, and cold temperatures may interfere with and be a hindrance to operational and maintenance activities, the spillway gate operating machinery should be placed inside an enclosure. The use of a house with enough room to access all parts of the machinery is usually advisable. The house may be constructed of metal (steel or aluminum) or reinforced concrete. The reinforced concrete houses require less maintenance and can be designed to be more aesthetically pleasing than the metal houses. Adequate ventilation and electrical illumination will need to be provided. In warmer climates, the spillway gate machinery may either be enclosed by a metal cover or be left in the open air with no cover except for parts susceptible to weather damage. These parts can be covered by shrouds. The designer, in conjunction with operations personnel, will need to decide which of the above options is most desirable for a specific project. However, it is usually a good idea to have similar machinery treatment for all dam projects on the same waterway. A typical concrete machinery house is shown in Plate 20.

b. Design loading. When machinery houses are made of either metal or reinforced concrete, the design live loading due to wind, snow, and ice must be considered for the area where the project is located. The roof should be designed to withstand a live load of at least 30 psf over the entire roof area, plus the maximum live load expected on any overhead lifting hooks or traveling hoists attached to the roof ceiling. The thickness of the roof and walls of a reinforced concrete structure will usually be in the range of 6 to 8 in.

c. Requirements for machinery. For removal and maintenance access, all machinery houses must have removable panels with lifting attachments that can be handled by crane. Thus, the crane can lift off the removable panel and then lift out the heavy machinery, or any of its components, for extensive maintenance or repair. Where machinery covers are used in lieu of houses, lifting attachments will be necessary on the covers, so that a crane can lift off the entire cover should extensive maintenance or repair be necessary. Hinged panel openings should be provided on the equipment covers for normal inspection and maintenance activities. For ease of handling, equipment covers should be made of metal shrouds.

13-4. Line Hooks

Line hooks should be placed upstream and downstream at appropriate locations in the dam pier faces and adjacent lock wall faces for use in tying up the floating plant for maintenance and emergency activities. They are usually placed in a series, one directly above the other, about five feet apart starting a short distance above pool level. The line hooks are typically fabricated from 8-5/8-in. OD, 1-in.-thick wall steel tubing (ASTM A519, Grade 4130, condition SR) and filled with grout. Cast-iron or cast-steel hooks should not be used because it is becoming increasingly difficult to get quality castings. The line hook and anchorage should be designed for reactions resulting from a 160-kip line pull using normal allowable stresses. The arrangement should include a curved steel frame and anchorage reinforcement. See Figure 13-1 for line hooks details.

13-5. Check Posts

Check posts, suitable for use by the floating plant in tying up to the structures during maintenance and emergency activities, should be provided on the top surfaces of all piers both upstream and downstream. The check posts are typically fabricated from 8-5/8-in. OD, 1-in.-thick wall steel tubing, ASTM A519, Grade 4130, condition SR, and filled with grout. Cast iron or cast steel should not be used. Checks posts with embedment should be designed for a minimum line pull of 160 kips using normal allowable stresses. See Figure 13-2 for check post details.

13-6. Deadman Anchorage for Floating Plant

In case of uncontrolled flow through a spillway bay, because of blockage of the bay by a barge or other vessel or due to a machinery failure or gate malfunction, it is advisable to furnish a means for anchoring the floating plant above the dam. This can be accomplished by providing deadman anchorages on the banks above the dam and locating a check post(s) on the riverward side of the adjacent upper guide wall of the lock. The floating plant can then tie to these items and be reasonably stable for work in the flowing water to either place spillway bulkheads or remove the obstruction from the bay.
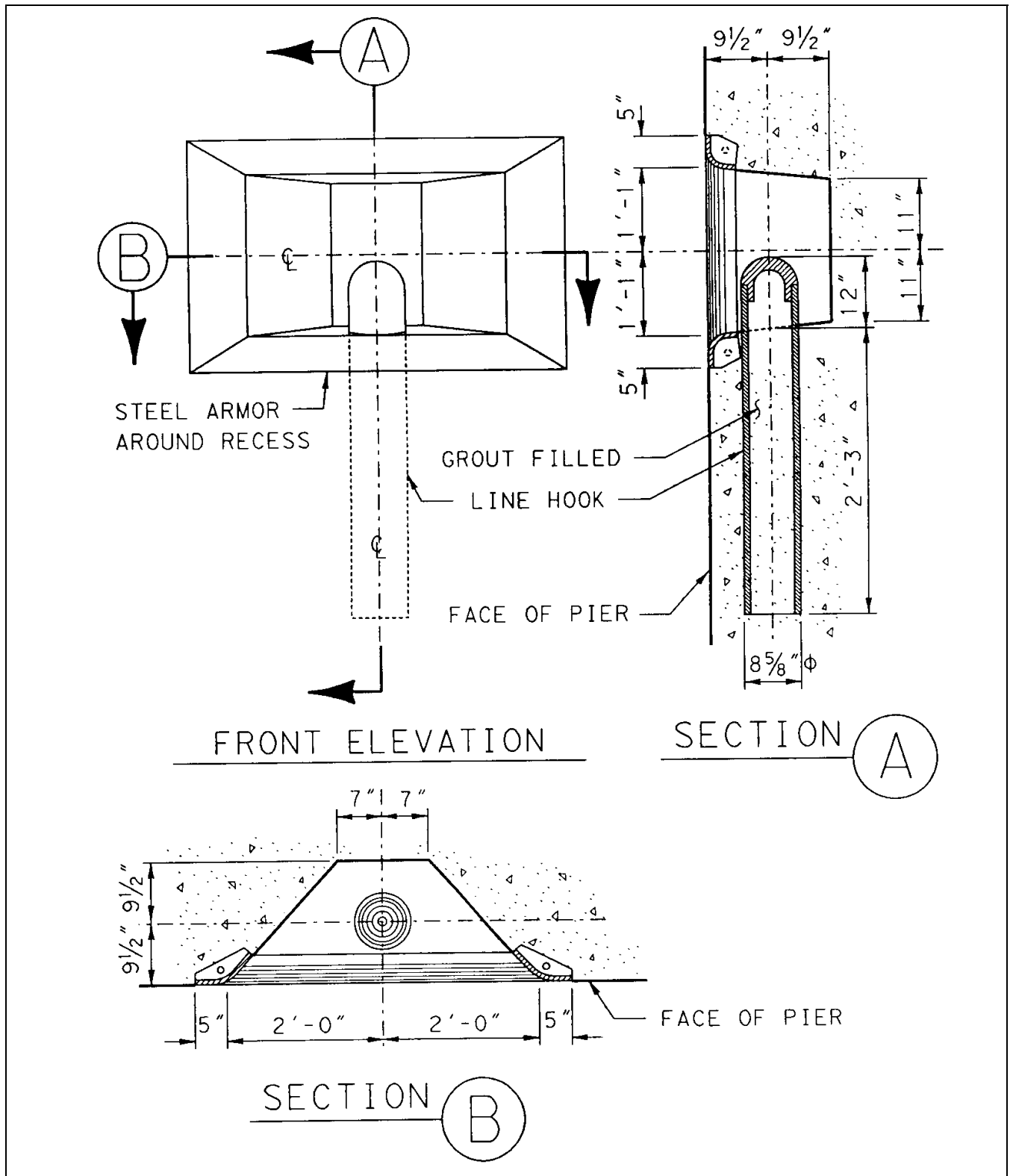
Figure 13-1. Line hook recess detail

ELEVATION

SECTION Ⓐ
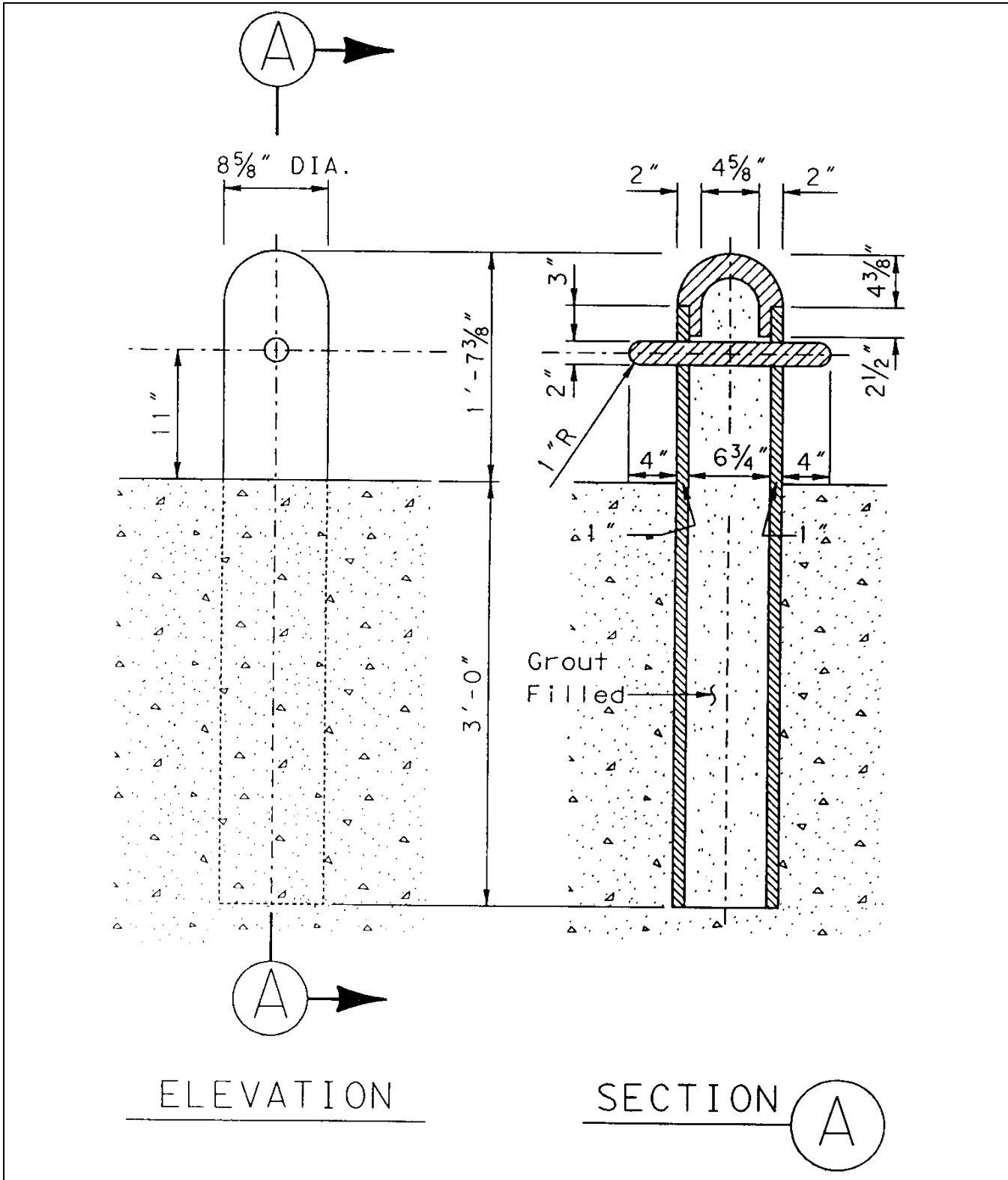
Figure 13-2. Check post detail

13-7. Ladders and Stairs

To provide safe and timely access for normal maintenance of and emergency attention to operating machinery and other important parts of the dam, permanent ladders or stairways should be provided in accordance with EM 385-1-1. Permanent ladders should be used only when the use of stairways is not practical or feasible. Stairs may be either concrete cast in place, precast concrete, or structural steel. Treads shall be provided with nonslip material at least at the nosing. Ramps, rather than stairs, may be provided when fewer than three risers are involved; the surfaces of such ramps should be covered with an abrasive material.

13-8. Access to Trunnion Area and Bulkhead Slots

Where practical, safe ladder or stair access should be provided to the top of all concrete trunnion girders whereby the tainter gate trunnion pin, yoke, and arms can be inspected and serviced. Piers will usually be wide enough to accommodate an interior stairway from the service bridge level down to a walkway opening on the downstream face of the pier for access to the top of the trunnion girder. Access to the lower part of the trunnion girder and prestressed anchorage covers will usually require a permanent ladder installation. Since the tops of the upstream bulkhead slots and bulkhead dogging devices are provided with a level walking surface, they can be accessed from this same interior stairway and another walkway opening. Access to the top of the downstream bulkhead recess may be possible only from the floating plant. However, permanently fixed ladders may be provided if practical.

13-9. Corner Protection

All bulkhead slots should be provided with structural steel corner protection on vertical upstream and downstream surfaces plus steel protection at the top of the slots. This will prevent damage to the slots due to bulkhead installation activities and from flowing water and the material that it carries. If vertical-lift spillway gates are used, the necessary recesses in the piers will also require similar corner protection. Corner protection may also be required at other locations depending on whether or not they are involved with maintenance or operational activities, subject to flowing water, or other potentially damaging situations.

13-10. Handrail and Guardrail

Permanently fixed handrails and guardrails should be provided, in accordance with EM 385-1-1, at all locations on a dam structure where safety and operational needs dictate their necessity, including the top deck of service bridges; the tops of concrete trunnion girders; stairways, inside or on sides of piers; walkway surfaces on piers, especially at the tops of bulkhead slots; and around vertical access shafts and ladder recesses and the arms of large tainter gates. Galvanized steel, aluminum, or painted steel handrails and guardrails may be selected for use. Anodized aluminum rails and posts will require a larger pipe diameter than steel, to meet strength criteria. Also, the strength reduction effects of welding must be considered in computing the required size of aluminum rails and posts.

13-11. Parapet Walls

Where conditions on a dam are such that concrete parapet walls are more desirable than handrails and guardrails, reinforced concrete parapet walls may be used as shown in Figure 13-3. The height of the wall should be the same as that of the metal rail, and the concrete thickness should be at least 8 to 12 in., with proper top slopes and possible aesthetic treatment.

13-12. Grating

Recesses, access shafts, catwalks, machinery platforms, and pits in the dam piers and at other locations in the dam should be provided with covers. Usually, galvanized steel grating is provided if grating is to be manually removed. In some cases, it may be necessary to cover the grating with steel plate for safety purposes.

13-13. Service Bridges

a. General. Service bridges provide support for overhead cranes and/or provide access to mechanical equipment located on the dam structure. Typically, traveling hoist cars or gantry cranes transport emergency bulkheads or vertical-lift gates to each gate bay and in some cases the navigation lock. The crane may also have an auxiliary crane attached for use in maintenance operations. A typical hoist car with auxiliary crane, supported by a service bridge, is shown in Plate 9. Plate 13 shows a cross section of a spillway service bridge for the gantry crane at John Day Dam on the Columbia River. A

smaller service bridge which serves as a personnel walk-way is shown in Plates 1 and 2. Plate 11 shows a vehicle access bridge across the dam spillways at Lock and Dam "D" on the Tennessee-Tombigbee Waterway.

b. Service bridge superstructure.

(1) For some of the larger dam projects on the Ohio and Mississippi Rivers, the service bridge usually consists of a concrete deck and up to three simply supported American Association of State Highway and Transportation Officials (AASHTO)- type precast, prestressed gird-ers (beneath each crane rail) that act as a composite system. The crane rail is centered on the middle girder. The clear opening between the upstream and downstream girder assemblies allows passage of the crane cab and transport of the bulkheads between the girders when the service bridge also serves the lock chamber. The bottom elevation of the girders is selected to allow a specified clearance above maximum operating upper pool for navi-gation through the lock. This specified clearance will usually be available from the U.S. Coast Guard and is the same clearance as is required for the low steel of railway and highway bridges which cross the waterway.

(2) Plate 5 shows a plan view of a service bridge that serves both the dam spillways and the lock chambers at the Melvin Price project on the Mississippi River. Plate 6 shows a cross section of this same service bridge. Plate 8 shows a cross section of a similar service bridge installa-tion at Smithland Locks and Dam on the Ohio River.

c. Service bridge substructure. The service bridge substructure is provided by the dam piers and piers located on the navigation lock walls. Reinforced elasto-meric bearing pads or rocker-type assemblies are recom-mended for bridge bearings. Cable restraints and concrete shear blocks should be provided to enable positive means of anchoring the superstructure to the substructure for seismic loads.

d. Crane rails. The service bridge crane rails should be sized to fit the crane wheel flanges. The 175#-per-yard American Railway Engineering Association (AREA) rail has been found suitable for projects having heavy hoist cars that travel with heavy sectional bulkheads and lower several sections latched together onto the spillway sill. The base of the rail should rest on heavy bearing plates at the service bridge deck level with embedded anchor bolts and levelling nuts for setting the crane rail to the proper elevation. A continuous plate may be required to reduce concrete bearing stresses to an acceptable level. These anchor bolts should extend through this plate and

through rail clips which anchor the rail in place. Careful attention should be given to sizing, detailing, and locating the expansion and contraction joints in the rail, and to corrosion mitigation of embedded anchor bolts. Standard AREA splice bars should be used at all rail splice loca-tions. Bituminous material can be used to cover the rail splice bars and the protruding anchor bolt heads and clips. Care must be taken to ensure against water being trapped and corroding plate anchor bolts. Details of the rail splices and rail anchorage assembly without bituminous coverings are shown in Figure 13-4.

13-14. Structural Instrumentation

Instrumentation for securing structural data in a dam structure may yield the following information: uplift pres-sures, concrete monolith tilt or differential movement, steel sheet pile cell interlock tension or cell movement, anchorage tendon or rod tension stress retention, concrete crack width increase, pore pressure, interior concrete temperature, leakage, and alignment monumentation observations. EM 1110-2-4300 provides adequate cover-age and guidance for most of the required structural instrumentation.

13-15. Warning Signs

Signs reading "Danger-Stay-Away," or "DANGER-STAY _____ FEET-AWAY-FROM-DAM," or some other appropriate message should be mounted on the upstream and downstream handrail (or parapet) of the dam service bridge in the middle of the spillway and facing toward the upper and lower pools. Both the frame for the signs and the background for the letters should be of steel material designed to withstand a minimum of 30-lb/sq-ft wind pressure. The lettering should be a red color with a con-trasting background and the individual letters should be 24 in. high or more and approximately 12 in. wide so that the sign can be read by a person with normal vision from a safe distance suitable for the specific project conditions. The signs should have adequate lighting so they can be easily read at night from this same distance. Refer to ER 1130-2-306.

13-16. Embedded Metals

a. General. Certain navigation dam steel structures will require embedded metals that should be installed in second-pour blockouts. Use of the second placement procedure allows for proper positioning of the embedded item at the designated grade and alignment for proper functioning of the companion structure. See Plate 19 for

SLOPE

SLOPE

2'-0"

DRAIN DETAIL PLAN

1'-6½"

5" 1'-1½"

6¾ 6¾" 2"

1½"

3'-6"

2½"

12"

¼"

SLOPE

STEEL SLEEVE
4" x 3"

2'-0"

3⅜
12"

DRAIN SECTION (A)
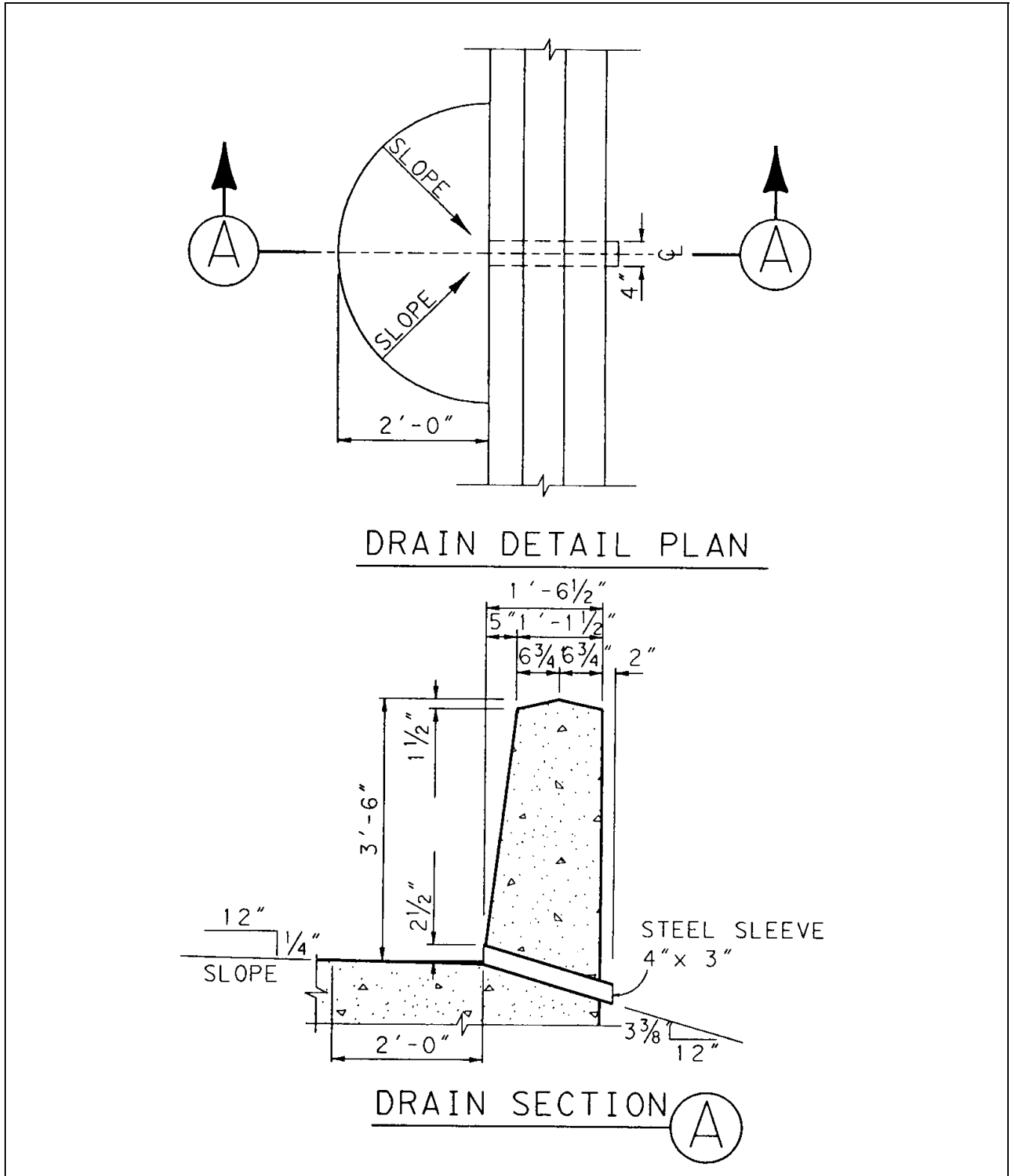
Figure 13-3. Concrete parapet detail

RAIL SPLICES

RAIL ANCHORAGE ASSEMBLY
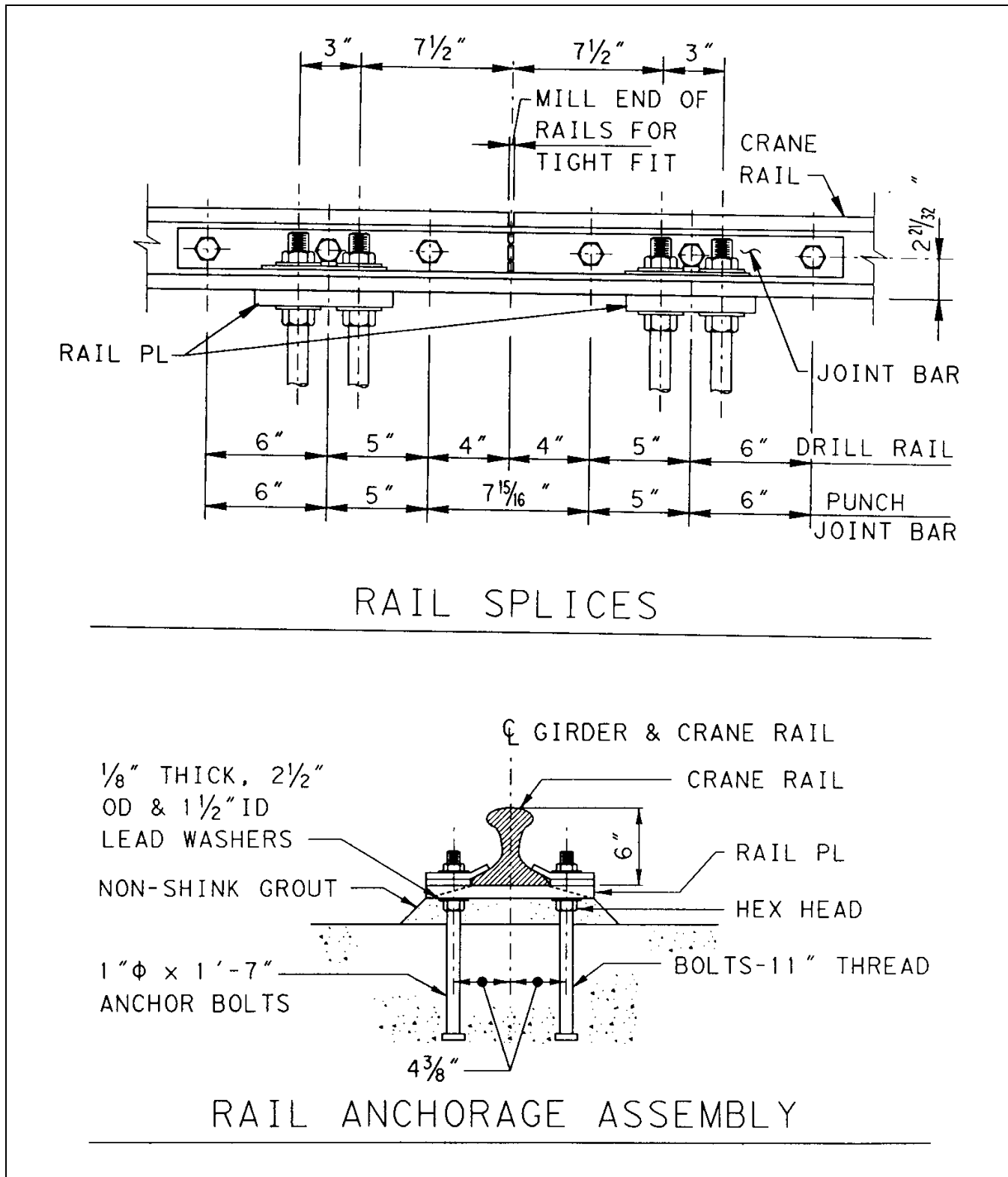
Figure 13-4. Rail anchorage assembly

typical embedded structural steel members with anchorages and adjustment bolts.

b. Embedded items and function. The primary embedded structural steel items and their usage are as follows.

(1) A curved side seal and rubbing plate are used with rubber "J" seal mounted on each side of spillway tainter gate.

(2) A continuous sill beam serves as level seating and metal-to-metal sealing surface for spillway gates--tainter, vertical-lift, or roller--and spillway bulkheads.

(3) Individual sill bearing plates serve as level seating surfaces for spillway bulkheads.

(4) Recess (slot) armor plates, bearing plates, seal plates, and guide plates for vertical-lift spillway gates and spillway bulkheads serve as true vertical bearing surfaces to transfer gate or bulkhead water load to concrete; serve to seal in conjunction with rubber "J" seal on each end of gate or bulkhead; serve to protect concrete in and adjacent to the gate or bulkhead recesses (slots) from damage during installation and removal activities; and serve as contact rolling and positioning surfaces.

(5) The trunnion anchorage assembly is used with prestressed concrete trunnion girders for tainter gates.

(6) The gate hoist machinery frame (exposed frame with embedded anchor and adjusting bolts and grout support) serves as a level base and support for hoist machinery and components.

(7) Service bridge embedded metals include crane rail anchorage and hand rail anchorage.

(8) Other miscellaneous embedded steel items are used for bulkhead dogging devices, concrete corner protection, grating supports, handrail supports, stair treads, ladder supports, tainter gate stops, and mooring rings and check posts.

c. Material. The most durable and long-lasting materials must be chosen for embedded metals--especially for metals that are continuously submerged--such as anchor bolts and nuts, horizontal seal and sill plates, curved and vertical seal and bearing plates, recess (slot) armor and guide plates, wicket gate components, drum gate components, hinged-crest gate components, etc. Embedded items whose surfaces serve only for armoring protection

and/or positioning purposes can be made of ASTM A-36 steel and painted, whether exposed or submerged. The embedded portions of steel items are never painted. However, the embedded portions need thorough cleaning to remove mill scale, rust, and dirt prior to concrete placement. This will help to ensure a good bond of the steel and concrete. Aluminum must not be embedded in concrete without some sort of separation coating, such as bitumastic, on the embedded portion. Embedded items with surfaces that serve as rubbing surfaces, guide surfaces, seal surfaces, or bearing surfaces, whether exposed or submerged, must be made of either solid stainless steel or clad stainless steel.

13-17. Mechanical and Electrical Features

a. General. This section will provide a broad overview of mechanical and electrical equipment with a brief description of the functions of the various items.

b. Mechanical and electrical. The major mechanical and electrical features for a navigation dam include the following:

(1) Hoist machinery for spillway gates. As an example, the gate hoists for a tainter gate will consist of two fixed units, one at each end of the gate, located near the top of the piers. These units will be driven by one electric motor mounted on one of the units. The drive unit and the driven unit will be coupled together by a line shaft or a torque tube extending between the two units. The drive side will have the electric motor, two bull gears, speed reducers, cable drum, and a brake. The driven side will have two bull gears, speed reducers, and a cable drum or chain rack. All these items will be mounted on a structural steel frame. The electric motor horsepower required will be determined by the requirements of EM 1110-2-2702. The operation of the individual gate hoist motors will be controlled by a pushbutton-type master control station located near each drive unit. Remote operation of the hoists can also be provided. For complete details of hoist mechanical and electrical features, see EM-1110-2-2702. See Plate 20 for a typical layout of hoist machinery. In Europe, gates are raised or lowered with hydraulic cylinders instead of cables. This type of design should be considered.

(2) Operating machinery for wicket gates and hinged-crest gates. Typical machinery will usually be hydraulic and will consist of a hydraulic power unit, hydraulic cylinder, operating rod, linkage to structure, and torque tube. One hydraulic pump may serve one or more cylinders with valving that would allow directing pressure

to other cylinders in the system. The Olmsted wicket dam on the lower Ohio River is an example of this type of hydraulic system. Dam No. 3 on the Red River in Louisiana has a single hinged-crest gate, which is operated by a hydraulic cylinder arrangement at each end. Electric power must be provided to all hydraulic pumps.

(3) Traveling gantry crane. Traveling gantry cranes are sometimes used for installing and removing vertical spillway gates, vertical bulkheads, and sectional bulkheads. Necessary electric power is supplied to the gantry crane by either power takeoff rails or a retractable power cable on a reel. Procurement of this crane will be by a dimensional requirement drawing and a performance specification.

(4) Traveling hoist car. The traveling hoist car is used for transporting, installing, and removing spillway sectional bulkheads. Electric power is supplied to the hoist car via power takeoff rails. Procurement is by the same method as for the gantry crane.

(5) Sluice gates and operators. Operators may be needed for raising and lowering sluice gates. A limited number of sluice gates are used on navigation dams. When they are provided, they will be used for minimum-flow water quality releases, discharges to attract fish, or drawdown of the reservoir in an emergency situation. Electric motors are used for operating power. Some large sluice gates will be hydraulically operated. Also, large sluice gates are usually installed in pairs on a single conduit for operational safety, inspection, and maintenance purposes. Upstream bulkhead slots may also be provided.

(6) Sump pumps. Sump pumps are used for pumping water from the drainage gallery sump and discharging the water into the lower pool.

c. Effects on concrete structures. The above items influence the concrete structure layout and design requirements in a variety of ways.

(1) A tall pier is required for the spillway gate hoist machinery in order to operate the gate through its designed range of travel. Also, it is desirable to provide a machinery house or cover in certain cold climates. The line shaft connecting the hoist machinery on adjacent piers can often be supported by brackets or cantilevered steel members fastened to the service bridge or to catwalk framing. If the layout is not suitable for this arrangement, a self-supporting torque tube must be used in lieu of the line shaft.

(2) The wicket gate operating machinery needs to be located in a watertight gallery. The hinged-crest gate machinery needs special support and operating space at each end of the gate for housing of hydraulic cylinders and crank mechanisms.

(3) Both the traveling gantry crane and traveling hoist car need a high service bridge with crane rails to travel on for performing spillway gate and bulkhead installation and removal procedures.

(4) The sluice gates and operators require a water-carrying conduit through the dam. Also required are sluice gate access galleries, operating rooms, shafts, and a bulkhead shaft extending to the top of the dam.

(5) The gallery pumping system must have gutters in the drainage gallery which collect and direct water to a sump pit adjacent to the gallery room containing the pump. There must also be a pump discharge pipe to the lower pool.

13-18. Catwalks

Structural steel personnel catwalks will need to be provided on navigation dams when there is no other practical means of access to project features that require periodic inspection, adjustment, greasing, painting, or other maintenance as well as possible replacement. Galvanized steel framing, handrail, and grating will normally be used. Design live loading should be at least 100 lb/sq ft with localized loads of the heaviest equipment or machinery expected possible on the structure.

13-19. Dam Lighting

Lighting facilities need to be provided for use by operating personnel and maintenance, inspection, and emergency crews, and for the benefit of navigation interests. Lighting will be required for the following areas: stairwells in bridge piers, galleries and access shafts, service bridge roadway, piers (exterior), machinery house interiors, spillway gate bays, crane and hoist cabs, fixed weir, signal lights, and warning signs. Reference ER 1130-2-306 and EM 385-1-1. Enclosed spaces such as stairwells, galleries, access shafts, crane and hoist cabs, and machinery houses need to have good lighting for use by operating, inspection, and maintenance personnel. Switches should be conveniently located and readily available at adits so as not to have to be searched for in the dark. Some type of additional low-level emergency lighting could be considered for galleries. Service bridge roadway lighting can

be pole-mounted on the machinery house at each pier. If a machinery house is not used at the service bridge roadway level, the lighting can be mounted either on the roadway guardrail or on poles. Exterior pier lighting will include upstream and downstream floodlights and floodlights on each pier face for lighting the spillway bays. Upstream and downstream signal lights should be mounted as high as possible on the dam structure. The number required will depend on the specific project layout. Upstream- and downstream-oriented searchlights should be mounted high on the piers or bridge roadways at locations readily accessible to operating personnel. The number required will depend on the specific project requirements. Some navigation dams have fixed overflow weirs which need to have some lighting. This can be achieved by a floodlight on the face of the adjacent pier at one end and a pole-mounted floodlight at the other end.

13-20. Lightning Arrestor System

A sufficient-size grounding cable should be provided to connect all machinery, electrical apparatus, conduits, conduit supports, crane rails, hand railing, spot and floodlights, lighting standards, and all extensive exposed metal items to a ground mat or ground rods for protection of equipment and personnel. Some projects have also provided for protection of the dam and its metal features from potential direct lightning strikes. The lightning protection system is composed of several tall poles located on the highest point of the dam structure and connected by metal cables which are tied into the above grounding system. This type of lightning protection system was used on some of the Red River Waterway projects in Louisiana.

13-21. Cathodic Protection

All major structural steel structures which are submerged need to be provided with cathodic protection such as sacrificial anodes or the impressed current type to prevent corrosion. Submerged recess armoring, bearing plates, seal plates, etc., need to have cathodic protection when they are composed of dissimilar metals--stainless steel in combination with carbon steel, for example. When steel sheet piling is used as part of a fixed weir or other permanent structure, some type of protection should be provided at the water line, where the piling is constantly subjected to wetting and drying due to wave action. Some height of this piling, above and below the water line, should have corrosion protection. If cathodic protection is not used,

then a durable coating such as bitumastic or an increased steel thickness should be used.

13-22. Surveillance Systems

Television transmitters with appropriately located television screens (receivers) should be provided on dams so that the facility may be operated safely and efficiently. In planning surveillance systems, the designer should consult persons with experience and expertise in this technology--especially the latest state-of-the-art equipment available. For convenience of maintenance, the equipment chosen should be of standard types and reliable design. Choosing similar standardized equipment for all dams in a navigation system will allow for interchange of parts and equipment and reduce the supply of spare parts and equipment required in inventory. Television surveillance is especially useful in cases of remote operation of the spillway gates when recreational boats and fishing boats are likely to be in the water either upstream or downstream of the dam. Television surveillance will usually be supplemented by warning sirens or horns plus warning signs.

13-23. Waterstops

Waterstops prevent the migration of water through joints of dams. A double line of waterstops should be provided near the upstream face of the dam at all joints. For gated spillway sections the tops of waterstops should terminate at gate sills and tie in to embedded steel if provided. If a sheetpile cutoff is provided below the monolith, the waterstop should tie in to the sheetpile at monolith joints. A single line of waterstops should be placed around all galleries and other openings crossing monolith joints. Further guidance in the selection and use of waterstops can be found in EM 1110-2-2102.

13-24. Joint Materials

Dam structures are subject to volume changes due to temperature, moisture content, and chemical reaction. Adjacent monoliths may experience differential movement at joints due to exterior loading. To minimize these effects and preserve the integrity and serviceability of the structure, joints should be provided. The introduction of joints creates openings which must be sealed. Typical joint filler materials consist of a variety of substances and configurations, depending on the purpose of the filler. Detailed guidance in the selection and use of joint material can be found in EM 1110-2-2102.

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

RETURN WALL

RETAINING WALL

DOWNSTREAM GUIDEWALL

TAINTER VALVE

GATEBAY

DISCHARGE

CHAMBER MONOLITHS

FLOW

TRAINING WALL
SEE PLATE 15

TAINTER GATE MACHINERY HOUSES
SEE PLATE 20

RETAINING WALL

RETURN WALL

GUARD WALL

GATEBAY

TAINTER VALVE

MITER GATES
NOT SHOWN

INTAKE

TRAINING WALL
SEE PLATE 16

STILLING BASIN

SEPARATION WALL

SPILLWAY WALKWAY
BRIDGE

TAINTER GATES
NOT SHOWN

CREST GATED SPILLWAY

5-60' WIDE TAINTER GATED SPILLWAY

UPSTREAM GUIDEWALL

WALKWAY BRIDGE

CUTOFF WALL
SEE PLATE 16

LOCK & DAM No. 4
COUSHATTA, LOUISIANA

DAM DESIGN
RED RIVER WATERWAY - LOUISIANA
LOCK & DAM NO.4

PLATE 1

SPILLWAY WALKWAY BRIDGE

FOR MACHINERY HOUSE, SEE PLATE 20

MACHINERY PLATFORM

EL.162.50

DECK EL.138.00

SPILLWAY BRIDGE SEAT
EL.155.25

€ TRUNNION

ACCESS STAIRS & RAILING

PRESTRESSED CONCRETE TRUNNION GIRDER

CHECK POST

EL.126.50

EL.123.79

EL.121.70

EL.118.00 — € TRUNNION

POOL EL.120.00

LINE HOOK RECESS

SPILLWAY MAINTENANCE
BULKHEAD SLOT

EL.109.00 — CHECK POST

EL.103.00

SPILLWAY TAINTER GATE

DRAINAGE MANHOLE

TAILWATER EL.85.00

SPILLWAY MAINTENANCE
BULKHEAD SLOT

FLOW

EL.85.00

$x^2 = -40Y$

SILL

KEYED JOINT

EL.75.00

EL.70.00

EL.69.00

EL.62.00

EL.60.00

12" EMBEDMENT TYPICAL

6" DIA. STAINLESS
STEEL WELL PIPE

EL.54.00

EL.52.00

2.0' FILTER MATERIAL

6" DIA. STAINLESS
STEEL WELL PIPE

8" DIA. SCH. 80
PVC EQUALIZER PIPE

2.0' FILTER MATERIAL

UNDERDRAIN SYSTEM

UNDERDRAIN SYSTEM

130'-0" PIER MONOLITH

108'-0" STILLING BASIN MONOLITH

STEEL SHEET PILE CUTOFF

STEEL SHEET PILE CUTOFF

STEEL SHEET PILE CUTOFF

TYPICAL SPILLWAY BAY SECTION

SOIL SUPPORTED MONOLITH

DAM DESIGN

RED RIVER WATERWAY
LOCK & DAM NO.4
SPILLWAY CROSS SECTION

PLATE 2

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
LOUISVILLE, KENTUCKY

STEEL SHEET PILE WALLS
SEE PLATE 15

STEEL SHEET PILE WALL
SEE PLATE 16

STEEL SHEET PILE CELLS

DOWNSTREAM RETAINING WALL - STEEL SHEET PILE CELLS
SEE PLATE 15

360' DOWNSTREAM STONE PROTECTION LIMITS

STONE PROTECTION LIMITS
ADJACENT TO RIVERSIDE LOCK

60' UPSTREAM STONE PROTECTION LIMITS

STILLING BASIN

WICKET GATES

110' X 1200' RIVERSIDE LOCK

DAM - 2200' WIDE NAVIGABLE PASS SECTION
(RIVER MILE 964.4)

CONCRETE SILL

DOWNSTREAM APPROACH WALLS

FLOW
OHIO RIVER

110' X 1200' LANDSIDE LOCK

UPSTREAM APPROACH WALLS

STONE PROTECTION LIMITS

PICTORIAL VIEW OF PROJECT

OLMSTED LOCKS AND DAM

DAM DESIGN

OLMSTED
LOCKS & DAM

PLATE 3

74.59' SILL MONILTH

174.54' SILLING BASIN MONOILTH

FLOW

$\overline{G}$ AXIS OF HINGE

$\overline{G}$ GALLERY

10' NOMINAL WIDE WICKET GATE

TOP OF SILL
EL.278.0

EL.261.50

EL.258.50

EL.254.0

NAVIGABLE PASSAGE
PILING FOUNDATION NOT SHOWN

CUTOFF WALL

CUTOFF WALL

POOL EL.300.0

FLOW

10' NOMINAL WIDE WICKET GATE

WICKET SUPPORT STRUT (PROP)

TOP OF SILL
EL.278.0

MINIMUM TAILWATER EL.285.0

WICKET OPERATOR

EL.261.50

EL.258.50

STILLING BASIN EL.254.0

WET HYDRAULIC
CYLINDER CHAMBER

DRY GALLERY

NONNAVIGABLE PASSAGE
PILING FOUNDATION NOT SHOWN

CUTOFF WALL

CUTOFF WALL

DAM DESIGN

OLMSTED
LOCKS & DAM
SPILLWAY CROSS SECTIONS

PLATE 4

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ST. LOUIS, MISSOURI

ILLINOIS

NORTH
ILLINOIS ROUTE 3
SOUTH

WOOD RIVER LEVEE

VISITOR
OVERLOOK

VISITOR
PARKING AREA

EL 435.5

EL 435.5

EL 415

1 ON 4

1 ON 3.5

EL 380

UPSTREAM APPROACH WALL

RETAINING
WALL

110' X 600' AUXILIARY LOCK

₵ LOCK

DOWNSTREAM APPROACH WALL

CENTRAL
CONTROL
HOUSE

STONE PROTECTION LIMITS

PROTECTION CELLS

2-110' WIDE TAINTER GATE SPILLWAY BAYS

STONE PROTECTION LIMITS

UPSTREAM APPROACH WALL

110' X 1200' MAIN LOCK

₵ LOCK

LIFT GATE

DOWNSTREAM APPROACH WALL

PROTECTION CELL

PROTECTION CELL

STILLING BASIN

MISSISSIPPI RIVER

FLOW

A

STONE PROTECTION LIMITS

A

7-110' WIDE TAINTER GATE SPILLWAY BAYS

STONE PROTECTION LIMITS

NOTE:
SERVICE BRIDGE (WITH HOIST CAR)
ACROSS SPILLWAY BAYS & LOCKS

STEEL SHEET PILE CELLS-CUTOFF WALL

MISSOURI

PROJECT SITE PLAN

DAM DESIGN

MELVIN PRICE
LOCKS & DAM

PLATE 5

DAM AXIS C STA 0+00
EL 504.00
STAIRWAY SHAFT
SERVICE BRIDGE
EL 494.00
MACHINERY HOUSE
OPERATING FLOOR EL 481.00
EL 473.00
C TRUNNION
PRESTRESSED CONCRETE TRUNNION GIRDER
MOORING RING
EL 422.00
C TRUNNION
EL 417.00
POOL EL 419.0
MOORING RING
TAILWATER EL 395.00
SPILLWAY TAINTER GATE
MAINTENANCE BULKHEAD SLOT
EMERGENCY BULKHEAD SLOT
EL 379.00
$x^2 = -40y$
SILL
EL 373.00
EL 370.00
EL 365.00
CUTOFF WALL
120'-0" PIER MONOLITH
54'-8" STILLING BASIN MONOLITH
50'-0"
PAVING BLOCK MONOLITH
CUTOFF WALL

TYPICAL SPILLWAY CROSS SECTION
PILING FOUNDATION NOT SHOWN

STAIRWAY SHAFT
EL 494.00
SERVICE BRIDGE
MACHINERY HOUSE
EL 473.00
MOORING RING
110'-0" CLEAR
SO THE SAME BULKHEADS CAN BE USED FOR LOCKS & DAM
PRESTRESSED CONCRETE TRUNNION GIRDER
NOTE:
TAINTER GATE NOT SHOWN
MOORING RING
EL 379.00
UPSTREAM PIER VIEW
C PIER
45'-0"
SILL MONOLITH
DOWNSTREAM PIER VIEW
85'-0"
PIER MONOLITH

TYPICAL SPILLWAY PIER ELEVATION

DAM DESIGN
MELVIN PRICE
LOCKS & DAM
SPILLWAY CROSS SECTION & ELEVATION

PLATE 6

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NASHVILLE, TENNESSEE

₵ DAM AXIS

PARKING AREA

335

330

CUTOFF WALL

325

TAILWATER EL.300

335

330

325

POOL EL. 324

RIPRAP AREA

FIXED WEIR

RIVER MILE 918.5

OHIO RIVER

FLOW

SPILLWAY

11-110' WIDE TAINTER GATE

TRAINING DIKE

110' X 1200' RIVER LOCK

₵ LOCK

UPSTREAM APPROACH

110' X 1200' LAND LOCK

DOWNSTREAM APPROACH

EARTH DIKE

EARTH DIKE

POOL EL. 324

PARKING
AREA

TAILWATER EL.300

JDG CREEK

PROJECT SITE PLAN

DAM DESIGN

SMITHLAND
LOCKS & DAM

PLATE 7

123'-0" PIER MONOLITH

SERVICE BRIDGE & DAM AXIS ℄

EL. 412.75

MACHINERY HOUSE

EL. 400.75

EL. 400.75

EL. 391.00

EL. 371.75

EL. 364.12

PIER

TRUNNION GIRDER ANCHOR ZONE

℄ TRUNNION

MOORING RING

PRESTRESSED CONCRETE TRUNNION GIRDER

TAINTER GATE

POOL ELEV. 324.0

EL. 327.25

℄ TRUNNION

7'-6" RADIUS

MOORING RING

EMERGENCY BULKHEAD SLOT

TAILWATER EL. 300.00

MAINTENANCE BULKHEAD SLOT

EL. 290.00

$Y = \frac{X^2}{40}$

7'-6" RADIUS

SILL

EL. 277.00

EL. 265.00

EL. 265.00

EL. 271.00

KEY SUPPORTED

EL. 260.00

PIER MONOLITH OUTLINE

PIER MONOLITH OUTLINE

EL. VARIES

80'-6" SILL MONOLITH

33'-0"

63'-0" STILLING BASIN MONOLITH

FOUNDATION GROUT HOLES

DAM DESIGN

SMITHLAND
LOCKS & DAM
SPILLWAY CROSS SECTION

PLATE 8

MACHINERY HOUSE

SERVICE BRIDGE

AUXILIARY CRANE

FLOW

CRANE RAIL

BULKHEAD IN STORED POSITION

CRANE RAIL

SERVICE BRIDGE

₵ PIER

₵ PIER

₵ PIER

HOIST DRUM

BRIDGE CRANE
(HOIST CAR)

TYPICAL SPILLWAY SERVICE BRIDGE PLAN

20'-6"

10'-3"   10'-3"

4'-3" 3'-3"   3'-3"   6'-7"

10" DECK

CABLE TRENCH (TYPICAL)
ELECTRICAL RECESS

7'-0" GIRDER
SPILLWAY SPAN
& LOCK SPAN

AXIS OF DAM

6'-6"   7'-0"   7'-0"   6'-6"

14'-0"

CLEAR

TYPICAL SPILLWAY SERVICE BRIDGE SECTION

NOTE:
7'-0" GIRDERS ARE PRESTRESSED CONCRETE.
10" DECK IS CAST-IN-PLACE CONCRETE.

AUXILIARY CRANE

BRIDGE CRANE
(HOIST CAR)

RAILING OMITTED FOR CLARITY

SERVICE BRIDGE

MACHINERY HOUSE

BULKHEAD IN STORED POSITION

HOIST CABLES

LIFTING BEAM

110'-0" CLEAR
SO THE SAME BULKHEADS CAN BE USED FOR LOCKS & DAM

NOTE:
TAINTER GATE
NOT SHOWN

BULKHEADS IN POSITION ON SPILLWAY SILL

₵ PIER

₵ PIER

₵ PIER

TYPICAL SPILLWAY SERVICE BRIDGE UPSTREAM ELEVATION

DAM DESIGN

SPILLWAY SERVICE BRIDGE
PLAN, ELEVATION
& CROSS SECTION

PLATE 9

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MOBILE, ALABAMA

PROJECT SITE PLAN

NOTES:

1. BAY SPRINGS IS CONNECTED BY A
   CANAL TO THE PICKWICK PROJECT
   POOL ON THE TENNESSEE RIVER.
   THE TWO POOL LEVELS ARE THE
   SAME.

2. HIGH RESERVOIR STAGES ARE
   DIVERTED TO THE TENNESSEE
   RIVER THROUGH THE CANAL.

3. THE BAY SPRINGS PROJECT IS
   LOCATED IN MISSISSIPPI.

TYPICAL DAM CROSS SECTION

DAM DESIGN
TENNESSEE-TOMBIGBEE WATERWAY
BAY SPRINGS
LOCK & DAM

PLATE 10

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MOBILE, ALABAMA

EL.278

POOL EL.300

EL.310

MILE 398.4
DAM AXIS ℄
NON-OVERFLOW DAM

EL.310

EL.291

FLOW

110' X 600' LOCK

℄ LOCK

EL.278

POOL EL.300

EL.310

NON-OVERFLOW DAM

EL.310

EL.291

3-26" WIDE TAINTER GATE SPILLWAY

FLOW

EL.278

POOL EL.300

EL.310

NON-OVERFLOW DAM

EL.278

BRIDGE
PIER

SPLITTER WALL

TRAINING WALL

EL.254

EL.255.5

TAILWATER EL.270

EL.291

TAILWATER EL.270

EL.278

EL.257

℄ SPILLWAY

PROJECT SITE PLAN

DAM AXIS ℄

BRIDGE EL.315

FLOW

POOL EL.300

6' WIDE PIER

FILL
IMPERVIOUS

EL.270
EL.268

SLURRY CUTOFF WALL

MACHINERY DECK EL.315.67

TOP OF GATE
EL.301

EL.291

EL.285

TAINTER GATE

EL.295 - ℄ TRUNNION

EL.274.7

24" CAISSON PILE

58' PIER MONOLITH

6

5

12

12

TOP OF TRAINING WALL EL.274 (END PIER ONLY)

TOP OF SPLITTER WALL EL.270

EL.254

EL.256.5

178.5' SPILLING BASIN MONOLITH

100' CANAL PROTECTION

15'

TAILWATER EL.270

EL.255.5

EL.257

TYPICAL SPILLWAY CROSS SECTION

DAM DESIGN

TENNESSEE-TOMBIGBEE WATERWAY
LOCK & DAM "D"

PLATE 11

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
PORTLAND, OREGON

WASHINGTON

LONGVIEW

PROJECT SITE

JOHN DAY LOCK & DAM

ASTORIA

PACIFIC OCEAN

COLUMBIA

VANCOUVER

HOOD RIVER

THE DALLES

BONNEVILLE LOCK & DAM

THE DALLES LOCK & DAM

PORTLAND

WILLAMETTE R.

SALEM

OREGON

VICINITY MAP

86' X 650' NAVIGATION LOCK

DAM AXIS   ℄ LOCK

NON-OVERFLOW MONOLITH

LAKE UMATILLA

RIVER MILE 215.6

19-50' WIDE TAINTER GATE SPILLWAY

FISH LADDER

KLICKITAT CO.
SHERMAN CO.

POWERHOUSE

STATE BOUNDARY

FISH LADDER

W A S H I N G T O N

B.N.R.R. (S.P. & S.)

FLOW

COLUMBIA RIVER

U.P.R.R.

O R E G O N

N

PROJECT SITE MAP

RUFUS

DAM DESIGN

JOHN DAY
LOCK & DAM

PLATE 12

CRANE RAIL BEARING
DECK EL. 281.00
MACHINERY DECK
EL. 278.23
EL. 274.23
EL. 272.15
SPILLWAY BRIDGE
POOL EL. 260.00
TRUNNION ACCESS STAIRWAY
SPILLWAY STOPLOG SLOT
SPILLWAY TAINTER GATE
TRUNNION ACCESS DECK
FLOW
TRUNNION EL. 228.5
STEEL TRUNNION ANCHOR
$\frac{X^{1.61}}{45.855}$
EL. 210.00
X
Y
TRUNNION
$\frac{2}{3}$
EL. 152.00
EL. 153.03
$\frac{4}{3}$
50'-0" RADIUS
TAILWATER EL. 160.00
EL. 153.03
EL. 152.00
DRAINAGE & GROUTING GALLERY
EL. 120.00
EL. 112.07
EL. 114.00
MATCH LINE
EL. 112.07
₵ DAM AXIS

PIER NOSE DETAIL

SPILLWAY STOPLOG SLOT
PIER
$\frac{(X-13)^2}{635.04} - \frac{Y^2}{39.69} = 1$
₵ DAM AXIS

EL. 281.50
DECK EL. 281.00
EL. 272.15
CRANE RAIL BEARING
EL. 274.23
50'-0"
TYPICAL
EL. 210.00
₵ PIER
3V ON 2H
EL. 152.00
₵ PIER
3V ON 2H
EL. 112.07
DOWNSTREAM PIER VIEW
UPSTREAM PIER VIEW

TYPICAL SPILLWAY PIER ELEVATION

166'-4" PIER MONOLITH
190'-3⅜" STILLING BASIN MONOLITH

TYPICAL SPILLWAY CROSS SECTION
ROCK FOUNDATION

MATCH LINE
EL. 127.00
EL. 124.00
STILLING BASIN
EL. 114.00

DAM DESIGN
JOHN DAY
LOCK & DAM
SPILLWAY CROSS SECTION & ELEVATION

PLATE 13

ILLINOIS

FLOODWAY SPILLWAY

FLOODWAY NEEDLE DAMS

AUXILIARY LOCK

THIRD STAGE COFFERDAM

MAIN LOCK

FLOODWAY NEEDLE DAMS

SECOND STAGE COFFERDAM

MISSISSIPPI →   RIVER →

FIRST STAGE COFFERDAM

MISSOURI

EARTH FILL
EMBANKMENT

MELVIN PRICE LOCKS & DAM

DAM DESIGN

COFFERDAM LAYOUT
FOR
LOCK & DAM CONSTRUCTION

PLATE 14

CRANE RAIL ℄

℄ CRANE RAIL

EL. 281.00

FREE BOARD

MAXIMUM
POOL EL. 266.00

EL. 254.33

GROUTING AND DRAINAGE GALLERY

EL. 212.75

EL. 206.50

NON-OVERFLOW MONOLITH SECTION

SEE PLATES 3 AND 7

PS-TYPE STEEL SHEET PILING

R 12.46'

R 12.46'

R 32.10'

13.74'

13.74'

SELECT FILL

R 12.46'

79.88'

39.94'

℄ CELL

℄ CELL

STEEL SHEET PILE CELL PLAN

POOL EL.324.00

EL.326.00

CONCRETE CAP

2.5

1

SELECT FILL

TAILWATER EL.300.00

ROCK FILL

GROUND EL.260.00

EL.230.00

℄ CELL

FIXED WEIR

DAM DESIGN

TYPICAL NON-GATED
DAM SECTIONS

PLATE 15

PLAN

Z-TYPE PILING

EARTH FILL
OR ROCK FILL

ELEVATION VARIES

NATURAL GRADE
ELEVATION VARIES

ELEVATION VARIES

STEEL SHEET PILE WALL ELEVATION

ELEVATION VARIES

FREE BOARD

POOL ELEVATION

SPILLWAY

ELEVATION VARIES

TRAINING WALL SECTION

ELEVATION VARIES

FREE BOARD

POOL ELEVATION

TAILWATER ELEVATION

FILL

FILL

ELEVATION VARIES

CUTOFF WALL SECTION

¢ DIKE

ELEVATION VARIES

SPOIL ROCK FILL

3'-6" THICK
RIPRAP

NATURAL GRADE
ELEVATION VARIES

SHEET PILE
CUTOFF WALL

EARTH DIKE SECTION

DAM DESIGN

MISCELLANEOUS
DAM FEATURES

PLATE 16

SPILLWAY SECTION

WET PLACEMENT

DRY PLACEMENT

DAM PIER

STA.2+55.37 U.S.

STA.2+02 U.S.

COFFERDAM CELLS
(TO BE REMOVED)

ANGLE OF REPOSE
(ASSUMED 1 ON 1.5)
(TYP)

6'-GRADED STONE B

1 ON 1.5

STA.0+76.34 U.S.

EL. 375.0

STA.0+12.00 U.S.

DAM AXIS STA.0+00.0

EL. 379.0

DAM SILL

STILLING BASIN

1 ON 2.5

EL. 370.0

SUBAQUEOUS FILL

GROUNDLINE

6'-GRADED STONE B

PERVIOUS FILL

1 ON 2

EXCAVATION LINE

EL. 359.0

SHEET PILING

EL. 365.0

EL. 349.0

1 ON 2

BACKFILL

FILTER B

FILTER A

EL. 356.5

EL. 364.0

SHEET PILING

WET PLACEMENT

DRY PLACEMENT

STA.2+12.67 D.S.

STA.2+62.67 D.S.

1 ON 1.5

EL. 370.0

GROUND LINE
ELEVATIONS

72"-NATURAL ROCK CAPSTONE
15"-140- LB. TOPSIZE RIPRAP
6"-BEDDING ON GEOTEXTILE

DAM DESIGN

MELVIN PRICE DAM
STREAMBED PROTECTION DETAILS

PLATE 17

LEGEND

W_B - WEIGHT OF BRIDGE

W_C - WEIGHT OF CONCRETE

W_G - WEIGHT OF GATE

W_M - WEIGHT OF MACHINERY

W_W - WEIGHT OF WATER

D/S - DOWNSTREAM

U/S - UPSTREAM

DAM DESIGN

GATED DAM STABILITY
TYPICAL FREE BODY DIAGRAM

PLATE 18

BULKHEAD UPPER GUIDE SLOT

BULKHEAD LOWER BEARING SLOT

WHERE SEALING IS NO LONGER REQUIRED

WHERE SEALING IS REQUIRED

BULKHEAD OR GATE SEAL BEARING

KEY ELEVATION

PLAN - BULKHEAD BEARING

SECTION A

SECTION B

NOTES:
1. CRES/CLAD INDICATES CORROSION RESISTANT STEEL CLAD PLATE.
2. ALL PLATES, BARS, AND STEEL SHAPES SHALL BE CARBON STEEL (ASTM-A36).
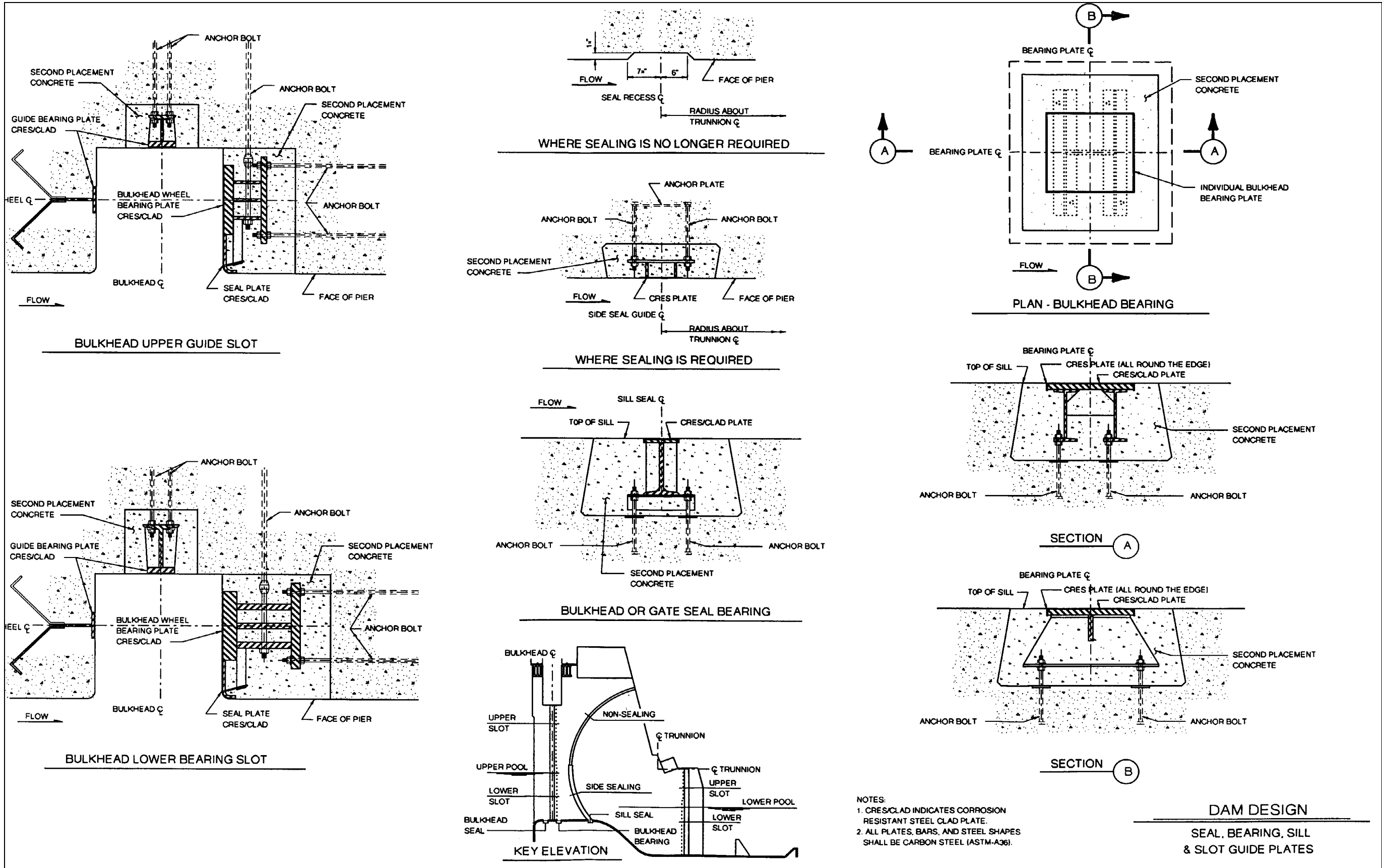
DAM DESIGN

SEAL, BEARING, SILL
& SLOT GUIDE PLATES

PLATE 19

PLAN

SPEED REDUCER SHAFT

SECONDARY REDUCER

REDUCER

MOTOR

COUNTER SHAFT

HOIST DRUM

SPEED REDUCER SHAFT

RIVEN SIDE

DRIVE SIDE

FLOW

PIER ℄

REMOVABLE COVER

PIER OUTLINE BELOW

STEEL SUPPORT FRAME FOR MACHINERY

DOWNSTREAM ELEVATION

PIER ℄

REMOVABLE COVER

CONCRETE OR STEEL PANEL WALLS & ROOF

WIRE ROPE BLOCKOUT

SIDE ELEVATION

REMOVABLE COVER

SPEED REDUCER SHAFT BLOCKOUT (10"~)

HOIST DRUM

WIRE ROPE BLOCKOUT

DAM DESIGN

MACHINERY AND MACHINERY HOUSE
FOR TAINTER GATE

PLATE 20

WATERSTOP

DAM SILL

12" PILING EMBEDMENT - TYPICAL

STEEL SHEET PILE CUTOFF

STEEL SHEET PILE CUTOFF

TYPICAL WATERSTOP LOCATION
(AT MONOLITH JOINT)

DAM SPILLWAY

DAM PIER

LOCK WALL

EXPANSION JOINT MATERIAL
(1" THICK X 12")

WATERSTOP

DAM SILL

UPSTREAM ELEVATION

NOTE:

WATERSTOPS MAY BE PVC, RUBBER, COPPER, OR STAINLESS STEEL.

STRUCTURE FACE

CONTR. JT.
MONO. JT.

3/4" CHAMFER

2 ND. PLACEMENT

1 ST. PLACEMENT

1'-0"
UNLESS OTHERWISE NOTED.

WATERSTOP

9"

WATERSTOP JOINT DETAIL

#5 REINF. BAR

9"

WIRE WATERSTOP AND #5 REINF.
BAR SECURELY TOGETHER AT 9"O.C.

1/2" SEALANT
BOND BREAKER

MONO. JT.
CONTR. JT.

STRUCTURE FACE

1/2" JT. FILLER

EXPANSION JOINT DETAIL

DAM DESIGN

WATERSTOPS AND
EXPANSION JOINT DETAILS

PLATE 21

Appendix A
References

A-1. Required Publications

PL 91-190, National Environmental Policy Act (NEPA)

National Historic Preservation Act

Public Law 99-662

TM 5-809-1
Structural Design Criteria for Loads

ER 1105-2-100
Guidance for Conducting Civil Works Planning Studies

ER 1110-2-1404
Hydraulic Design of Deep Draft Navigation Projects

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

ER 1110-2-1461
Design of Navigation Channels Using Ship-Simulation Techniques

ER 1110-2-1806
Earthquake Design and Analysis for Corps of Engineers Projects

ER 1110-2-8152
Planning and Design of Temporary Coffer Dams and Braced Excavation

ER 1130-2-306
Navigation Lights, Aids to Navigation, Charts, and Related Data Policy, Practices and Procedures

EP 25-1-1
Index of Publications

EM 385-1-1
Safety and Health Requirements Manual

EM 1110-1-1904
Settlement Analysis

EM 1110-1-1905
Bearing Capacity of Soils

EM 1110-2-1601
Hydraulic Design of Flood Control Channels

EM 1110-2-1602
Hydraulic Design of Reservoir Outlet Works

EM 1110-2-1603
Hydraulic Design of Spillways

EM 1110-2-1604
Hydraulic Design of Navigation Locks

EM 1110-2-1605
Hydraulic Design of Navigation Dams

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

EM 1110-2-1612
Ice Engineering

EM 1110-2-1901
Seepage Analysis and Control for Dams

EM 1110-2-1902
Stability of Earth and Rock Fill Dams

EM 1110-2-2000
Standard Practice for Concrete for Civil Works Structures

EM 1110-2-2102
Waterstops and Other Joint Materials

EM 1110-2-2104
Strength Design for Reinforced-Concrete Hydraulic Structures

EM 1110-2-2200
Gravity Dam Design

EM 1110-2-2300
Earth and Rock-Fill Dams General Design and Construction Considerations

EM 1110-2-2400
Structural Design of Spillways and Outlet Works

EM 1110-2-2502
Retaining and Flood Walls

EM 1110-2-2503
Design of Sheet Pile Cellular Structures Cofferdams and Retaining Structures

EM 1110-2-2602
Planning and Design of Navigation Lock Walls and Appurtenances

EM 1110-2-2701
Vertical Lift Crest Gates

EM 1110-2-2702
Design of Spillway Tainter Gates

EM 1110-2-2901
Tunnels and Shafts in Rock

EM 1110-2-2906
Design of Pile Foundations

EM 1110-2-3506
Grouting Technology

EM 1110-2-4300
Instrumentation for Concrete Structures

EM 1110-8-1(FR)
Winter Navigation on Inland Waterways

ETL 1110-2-256
Sliding Stability for Concrete Structures

ETL 1110-2-338
Barge Impact Analysis

ETL 1110-2-365
Nonlinear Incremental Structural Analysis of Massive Concrete Structures

CW-02411
Guide Specifications for Steel Sheet Piling

Headquarters, U.S. Army Corps of Engineers 1986
Headquarters, U.S. Army Corps of Engineers. 1986. "Uniform Federal Accessibility Standards (UFAS)," Washington, DC.

Fletcher 1993
Fletcher, B. P. 1993. "Baldhill Spillway, Hydraulic Model Investigation," Technical Report HL-93-6,

U.S. Army Engineer Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, MS 39180.

Shore Protection Manual (SPM) 1984
Shore Protection Manual (SPM). 1984. 4th ed., 2 Vols, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, U.S. Government Printing Office, Washington, DC.

U.S. Army Engineer District, Pittsburgh 1986
U.S. Army Engineer District, Pittsburgh. 1986. "After Action Report - Monogahela River Barge Breakaway Incident - November 1985," 1000 Liberty Ave., Pittsburgh, PA 15222.

U.S. Army Engineer District, Pittsburgh 1991
U.S. Army Engineer District, Pittsburgh. 1991. "After Action Report - Monogahela Barge Breakaway Incident - January 1990," 1000 Liberty Ave., Pittsburgh, PA 15222.

A-2. Related Publications

ER 1110-8-2(FR)
Inflow Design Floods for Dams and Reservoirs

EM 1110-2-1906
Laboratory Soils Testing

EM 1110-2-1907
Soil Sampling

EM 1110-2-1908
Instrumentation of Earth and Rock-Fill Dams (Part 1)

EM 1110-2-1908
Instrumentation of Earth and Rock Fill Dams (Part 2)

EM 1110-2-1911
Construction Control for Earth and Rockfill Dams

EM 1110-2-2105
Design of Hydraulic Steel Structures

EM 1165-2-303
Conservation Pools in Reservoir Projects

ETL 1110-2-352
Stability of Gravity Walls, Vertical Shear

Appendix B
Design and Construction Planning

B-1. Introduction

Information provided in Appendix B of EM 1110-2-2602 contains guidance on structural and project engineering responsibilities as well as details of the life cycle project management process as applied to a large scale civil works project. This guidance is also applicable to navigation dams. A reiteration of the structural and project engineering responsibilities is provided below.

B-2. Structural and Project Engineering Responsibilities

a. General. The Corps of Engineers operates in partnership with the Inland Waterways User Board (IWWUB), which shares the cost of designing and building navigation lock projects. Increased emphasis is being place on the key roles of the structural engineer (SE) and the project engineer (PE) in achieving high-quality products on schedule and within budget. To facilitate project development, a project management office was established in HQUSACE and in each division and district. In the project management system, the project manager (PM) is the primary point of contact for project coordination between the local sponsor and the Corps. The PM manages the project scope, schedule cost, and budget, and facilitates the resolution of existing or potential problems. The PM is also responsible for reporting the project's status to higher authorities and the local sponsor. This guidance should enable the SE and PE position to better support the PM. An important link in the cooperative relationship between engineering division (ED) and the PM is the individual designated as the PE. This individual should be a registered professional engineer and should be an SE in the case of a navigation lock project. However, the PE could come from other technical disciplines in ED. The PE position should not be filled by any other technical manager (TM). One PE is assigned to the overall project. The PE's role on the design team is to assist in the technical management of the project. Because a successful PE generally requires broad design experience and technical leadership, such assignments must be made to senior designers who have been delegated authority to perform their interdisciplinary responsibilities.

b. Structural engineering responsibilities. Structural design is a creative process that generally begins with a vague definition of the client's problems and proceeds to

a practical solution using basic engineering principles and modern technology. A navigation lock project is executed by a multidisciplinary team that may include several structural engineers. The SEs must determine the appropriate level of analysis required for each phase of the life cycle process. The analysis performed during the reconnaissance phase relies on engineering judgment with abridged numerical modeling. During this phase, the SEs will examine a minimum number of possible structural solutions that are mutually acceptable to local and federal interests. In contrast, during the preconstruction engineering and design phase, detailed analyses will be performed and structural features designed to the degree necessary to prepare quality contract documents. During the construction phase, the only analyses performed are those required to resolve field problems. Analyses during the operation and maintenance phase may be required for deficiencies, repairs, modifications, or replacement. The responsibilities for the structural design can be categorized as described below.

c. Design and analysis. One of the SE's primary responsibilities is to develop a structural solution that meets the design objectives. The SE will draw upon past experience to develop design concepts or examine new and innovative solutions. The SE will combine engineering judgment with engineering principles to develop a reliable basis for design. Depending on the project phase, the analyses may require manual computations that capture the general structural behavior or in-depth computer modeling using software developed by the Computer-Aided Structural Engineering (CASE) project.

d. Design quality. The engineering design team's performance influences the quality of each design phase. Quality is affected by the SE's ability to communicate, apply sound judgment, advise, plan, analyze, and review designs.

e. Cost estimating. The SE should help develop the cost estimate at each phase of the project. The level of this participation may vary; at whatever level, it is essential to developing a reliable cost estimate. (Public Law 99-662 limits projects authorized by the act to a 20 percent increase in the baseline cost, excluding increases due to inflation and changing legal requirements.) The SE should consider the reliability of engineering and other data available when developing contingency factors.

f. Design schedule and budget. Throughout the project's life, SEs should prepare and maintain their design schedules and budgets. This information is provided to the PE for preparing the engineering schedule

and budget. Individual schedules and budgets should be based on reliable data and information available from other disciplines. Contingencies should be included to account for uncertainties. An overly conservative budget or design schedule can result in termination of the study or require reauthorization of the project. On the other hand, overly optimistic estimates result in insufficient funding to complete the project. After the schedule and budget are approved, the SE should complete the design within that schedule and budget. Changes can be made only with written approval by the PM. Throughout the project, the SE should make comparisons between scheduled and actual progress, and budgeted and expended dollars, to assess performance. The SE should provide the results of these comparisons to the PE.

g. Technical coordination. The SE is involved in technical coordination of structural features during all project phases. The SE should coordinate structural design activities with individuals from other functional elements (geotechnical, hydraulic, mechanical, electrical, architectural, construction, operations, cost engineering, real estate, surveying, mapping, etc.) to develop the design of the structural features. Also, the SE should maintain technical coordination with the technical staff of the local sponsor. Technical coordination with a higher authority to reach early agreement on unprecedented or complex problems is encouraged.

h. Project engineering responsibilities. Selection of the PE is one of the most important management decisions for ensuring success. The PE should be a technical leader who has an overview of the project and a general understanding of the various functional elements and thus is able to support the PM by managing the design process.

(1) Management of design process. The PE, working with all appropriate disciplines, should define the engineering design objectives pertaining to customer care, innovation, engineering and design (E&D) costs, operation and maintenance (O&M) costs, modifications, quality, biddability, constructibility, and operability. The PE should identify the specific tasks required to support the design objectives and should integrate the team effort in an efficient and cost-effective manner. The PE should monitor team progress by reviewing the schedules and budgets and by measuring actual production, time elapsed, and funds expended. Changes should be documented and evaluated for impacts.

(2) Design quality. A quality product is the primary objective. Design quality is influenced by the PE's ability

to promote a team environment that encourages communication between engineering disciplines. The PE must recognize technical conflicts at an early stage. In addition, the PE should explore alternate designs that could improve quality or reduce costs.

(3) Design schedule and budget. The PE should coordinate and consolidate the budget and provide it to the senior engineering staff. Each discipline should prepare a detailed estimate to ensure that adequate resources are budgeted to perform all engineering functions for all phases.

(4) Cost estimate. The PE should ensure that quantities are being developed in accordance with the code of accounts so that no quantities are omitted or duplicated. Contingencies should be established and justified in terms of available information. The PE should inform the PM if additional engineering information is needed to reduce contingencies that have a significant impact on total cost. A sample template for making the cost estimate with typical-cost items for a navigation dam is provided in section B-4.

(5) Interaction with project manager. The PE should maintain a working relationship with the PM. It is essential that the PE inform the PM about the project status and contacts with the local sponsor. The PE should support the PM in developing the engineering aspects of the project management plan.

B-3. Design Checklist for Navigation Dams

a. Reconnaissance report. Structural analysis in this phase is usually limited to a few basic calculations used together with data from similar projects and proven engineering concepts to establish the project's viability.

(1) Approximate location and consideration or list of alternative sites.

(2) Preliminary determination of controlled and uncontrolled dam width.

(3) Need for navigation pass.

(4) Lift.

(5) Need for model studies.

(6) Preliminary foundation type (for cost purposes).

(7) Preliminary spillway configuration.

(8)   Overall structural layout of dam and appurtenant features that is a reliable base for the cost estimate.

(9)   Preliminary decision on type of gates (for cost purposes).

(10) Need for a bridge.

(11) Estimate of design effort required for feasibility report.

(12) Preliminary quantities for reconnaissance cost estimate.

(13) Estimate-design criteria.

(14) List of design memoranda.

(15) E & D cost estimate (input).

(16) Coordination with other elements as needed.

b. Feasibility report. A significant amount of structural analysis is accomplished during this phase. The design team examines and compares alternative solutions and then chooses the most suitable and economical solution. Sufficient structural analysis must be performed to ensure that the chosen solution is the appropriate one and that *quantities are reliable enough* to predict the construction cost within 20 percent.

(1) Finalize location.

(2) Determine   final   controlled/uncontrolled   dam width.

(3) Determine overall dam geometry:

  • Elevations of spillway crest, navigation pass, and bridge, length of spillway, etc.

  • Width of dam piers.

  • Type of gate and control elevations such as elevation of trunnion girder.

(4) Finalize type of foundation.

(5) Select type of lock gates based on functional requirements.

(6) Select type of filling and emptying system.

(7)   Establish requirements for operations buildings (control stands, etc.).

(8)   Decide on maintenance unwatering facilities.

(9)   Compute quantities for feasibility cost estimate.

(10) Establish preliminary design criteria.

(11) Establish list of guide specifications.

(12) Provide input for:

  • Outline of VE studies.

  • Detailed schedule and budget for E & D.

  • Project Management Plan (PMP).

(13) Describe special analyses or Feature Design Memoranda (FDM) required.

(14) Coordinate with other elements as needed.

c. Design memorandum (DM) phase. Most of the structural analysis will be performed in the DM phase. The design of all the representative structural components should be accomplished in the individual DMs as identified in the PMP. The DM should address the following issues:

(1) Need for detailed soil information.

(2) Final design criteria.

(3) Final loads.

(4) Loading conditions and critical cases.

(5) Final structural analysis of all gates, bulkheads, etc.

(6) Structural analysis and determination of areas of steel for all critical reinforced concrete members.

(7) Layout of dam accessories and embedded metals.

(8) Parapet-versus-hand-rail selection.

(9) Public access.

(10) Esplanade.

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(11) Control houses.

(12) Level of coverage for FDM presentation.

(13) Drawings to be presented in FDM and what will be shown on each.

(14) Preparation of FDM drawings.

(15) Proposed instrumentation relative to structural safety.

(16) Preparation of FDM text.

(17) Quantities for FDM cost estimate.

(18) Input for Critical Path Method (CPM) on materials and equipments.

(19) Coordination with other elements as needed.

d. Plans and specification phase (PED). In this phase, the team performs minimal structural analysis (main structural members) besides revisions needed to finalize the design. This phase consists mainly of designing the details, placing them on the drawings, and writing the specifications.

(1) Identify standard details to be used.

(2) List all drawings and what to present on each.

(3) Identify all details remaining to be designed.

(4) Prepare reinforcing steel layouts.

(5) Prepare contract drawings.

(6) Research previous similar jobs for specifications.

(7) Mark up guide specs and specs from other jobs.

(8) Write specifications as required.

(9) Prepare quantities for government estimate.

B-4

B-4. Detailed Cost Estimate

Cost Account	Item	Qty.	Unit	Unit Price	Total Cost
04	**DAM**				
	FOUNDATION				
	Excavation, common		cu yd		
	Excavation, rock		cu yd		
	Excavation, foundation		cu yd		
	Deep compaction of foundation fill		job		
	Fill, backfill		cu yd		
	Piling, steel bearing				
	HP 14x73, vertical		lin ft		
	HP 14x117, vertical		lin ft		
	HP 14x117, batter		lin ft		
	Pile tips for HP 14x73		each		
	Pile tips for HP 14x117		each		
	Steel, reinforcement (welded to tension pile)		lb		
	Pile load and driving tests		each		
	Piling, steel sheet (PZ22)		lin ft		
	Subtotal for Foundation				
	Contingencies				
	TOTAL FOR FOUNDATION				

Cost Account	Item	Qty.	Unit	Unit Price	Total Cost

04	**DAM, Continued**
	CONCRETE
	STRUCTURE
	Concrete, reinforced
	  Dam monolith bases
	  Dam piers
	  Service bridge piers
	  Non-overflow walls
	  Stilling basin
	Steel, concrete
	  reinforcement
	Waterstop
	Preformed joint filler
	
	Subtotal for concrete structure
	Contingencies
	TOTAL FOR CONCRETE
			cu yd
			cu yd
			cu yd
			cu yd
			cu yd
			cu yd
			lb
			lin ft
			sq ft		

	STRUCTURAL STEEL
	  Steel, tainter gates
	  Steel, emergency
	    bulkheads
	  Steel, maintenance
	    bulkheads
	  Steel, miscellaneous,
	    embedded
	  Line hooks and mooring rings
	
	Subtotal for structural steel
	Contingencies
	TOTAL FOR S. STEEL
			lb
			
			lb
			
			lb
			
			lb
			
			lb		

	SERVICE BRIDGE
	  Concrete
	  Concrete, precast I-Beams
	  Concrete Reinforcement
	
	Subtotal for Service Bridge
	Contingencies
	TOTAL FOR SERVICE
	  BRIDGE
			cu yd
			lin ft
			lin ft
			lb		

Appendix C
Lessons Learned - Case Histories

C-1. General

This appendix will review some of the major and minor problems that have been experienced on navigation dam projects designed and built prior to 1993. Also, references will be given to Corps districts and divisions and other Corps organizations involved and to published material which relates to a specific problem and the follow-up action taken.

C-2. Concrete

Several Corps districts have experienced problems with concrete durability in stilling basins and buckets, apparently due to below-average-quality concrete, insufficient reinforcing steel, and the abrasive action of ice, drift, gravel, stones, etc., carried in the spillway discharge water. The damage to the concrete has varied from surface abrasions to loss of enough concrete to expose the reinforcing steel, which originally had 4 in. or more of cover. The Pittsburgh and Nashville districts have experienced problems of this type. Not much can be done about the materials carried in the spillway discharges, but current engineering manuals have updated concrete quality (higher strength, more durable aggregates, etc.) and reinforcing steel guidance to prevent most problems of this type. Refer to Chapter 12 for further details. Since it is not possible, on most projects, to unwater the spillway bucket for repair without using extremely costly steel pile cofferdams and, during high flood flows, interfering with spillway discharges, it is highly desirable to follow the published design criteria and guidance for stilling basin concrete and reinforcing.

C-3. Spillway Tainter Gates

a. Use of submergible gates. When the first of the "current generation" Ohio River navigation dams were designed and built in the 1950s and 1960s, it was deemed advisable to use double-skin plated overflow tainter gates (submergible gates) for the purpose of passing ice and debris through some of the spillway bays at a project. The other spillway bays would also have similar double-skin plate gates, but they would rest on the sill and would not be submergible. Vibration problems, in addition to horizontal sealing problems at the sill, developed on the submergible gates from the beginning of their operation. It was also discovered that these gates were not satisfactory for passing ice and/or debris unless almost fully

lowered--a condition which could cause damage to the stream bed stone protection downstream of the stilling basin. After several years of unsuccessful attempts to solve the vibration and seal problems, the Corps adopted a policy of not using submergible tainter gates on future projects. Many of the submergible tainter gates remain in use on Ohio River navigation dam projects, but they are no longer operated as submergible gates. It was discovered that ice and debris could be passed satisfactorily through the spillways by raising the gates off the sill a sufficient distance to create enough discharge velocity to draw the ice and debris to and through the spillway bay underneath the gate. At Cheatham Dam on the Cumberland River in the Nashville district, the seven submergible tainter gates were modified to be nonsubmergible and to rest on a modified sill with a new ogee crest shape. The costs of the modifications at this project were almost balanced out by the revenue from additional power generated by savings in water losses (leakage) where the submergible gate failed to seal at the horizontal sill. The Huntington, Pittsburgh, and Louisville districts have background information on the problems and actions taken on the submergible gates on the Ohio River.

b. Passing of ice and debris. In addition to the submergible tainter gates described in paragraph C-3a above, the Corps has used "piggy-back" tainter gates on some projects in the Pittsburgh district. These gates are composed of an upper section and a lower section which operate independently of each other, the intent being to raise the upper section out of the water and thus allow ice and debris to pass over the lower section which remains in place on the sill. This arrangement has not proved satisfactory, however, and recent tainter gate designs have not used the "piggy-back" concept.

c. Cables (wire ropes) versus chains. Link chains for use in raising and lowering spillway tainter gates are composed of links, pins, spacer sleeves, spacer washers, racking collars, and retaining rings. The holes in the links and pins are machined to specific tolerances to allow easy movement of the link with relation to the pin. Links and pins are made of 4140 steel. Pins have been cadmium coated and chain-bearing surfaces coated with graphite lubricant during assembly. No grease grooves or grease fittings are provided for the pins; thus, all greasing of the pins and links has to be by manual application to exposed surfaces. These chains function well with no special problems for that portion that stays out of the water where normal maintenance and greasing can be done readily on a periodic basis. However, the portion of the chain that is continually submerged is subject to corrosion and pitting damage and can become less flexible and possibly

inoperative due to infrequent maintenance and greasing of the pins. The Nashville district experienced many operational and maintenance difficulties with the chains on the Cheatham Dam tainter gates, and eventually all the portions of the chains that were continually submerged had to be replaced. The replacement links and pins were made of AISI 4140 steel and had basically the same components except that copolymer washers were added on each side of all links. Also, each new pin was provided with three grease grooves and three grease fittings to allow lubrication of the pin-link contact surfaces. Chains were used exclusively until manufacturers developed a wire rope which could be wound on itself. These stainless steel wire ropes have proven to be much better from an operational and maintenance standpoint, and all new projects will use wire rope for tainter gate hoisting operations. For more information on stainless steel rope, see EM 1110-2-2702.

d. Treatment of bottom lip of tainter gates. Several Corps districts have experienced problems with the shape of the bottom lip of tainter gates, as well as problems with horizontal rubber seals used on the lip to provide a more positive seal at the sill.

(1) Shape of lip. The relatively broad shape of the lip of the gates at Barkley Dam on the Cumberland River in the Nashville district, coupled with an attached flat rubber sill seal and seal retainers, caused gate vibrations for a range of gate openings and tailwater levels. Investigations by the Nashville district, aided by WES, resulted in removal of the rubber seals and retainers and grinding of the bottom of the gate to obtain a satisfactory metal-to-metal seal. A report on this specific situation is available from WES. For new applications, emphasis is placed on making the lip of the gate as sharp as possible so that negative pressures, which can cause gate vibrations, are not created.

(2) Rubber seal on lip of tainter gate.

(a) The navigation dams on the Arkansas River in the Little Rock district had tainter gates which were equipped with horizontal rubber "J" seals, mounted on the gate lips for gate sealing at the sill level. Use of these "J" seals resulted in gate vibration problems when the gates were opened to pass water. These vibration problems were essentially solved by removing the seal and retainers. A WES report of its investigations and the resulting remedial actions is available. Experience and knowledge gained in the above two instances and from WES model

tests indicate that the use of rubber seals of any type on the lips of tainter gates could result in excessive gate vibrations.

(b) In most cases, an adequate metal-to-metal seal between the lip and sill can be obtained by proper adjustment (grinding) of the lip to precisely match the sill plate. However, if it is necessary to conserve every bit of water possible, a properly designed flat rubber seal assembly similar to that used on the Red River (Louisiana) dams tainter gates may be provided after proper coordination with and recommendations from WES and district hydraulic engineers. Rubber "J" seals should not be used at this particular gate lip location.

(3) Excessive leakage at lip of gate. The Cordell Hull navigation dam on the Cumberland River in the Nashville district has conventional tainter gates with ASTM A-36 steel lips that rest on embedded stainless steel sill plates. Because river flows are normally routed through the power plant adjacent to the spillway, the tainter gates are only off the sill when they are opened to pass flood flows. After the project had been in operation for several years, it was noticed that the gates had excessive leakage at the horizontal lip-sill contact. One gate was unwatered for inspection by using the spillway bulkheads, and a badly worn lip was revealed. The deterioration of the lip was judged to be due to lack of cathodic protection on the gates, and erosion-corrosion plus some cavitation. In order to remedy this situation, a stainless steel lip was installed on the gate and carefully adjusted for a better seal contact with the embedded stainless steel sill plate. Cathodic protection was also installed on the gates.

e. Side seals and rubbing plates for tainter gates. Molded rubber "J" seals have proven to function very efficiently as side seals for spillway tainter gates. However, when these seals were used with ordinary structural steel (ASTM A-36) embedded rubbing plates, the seals would frequently suffer damage if any undue offsets, irregularities, or heavy rusting was present on the steel rubbing surfaces. Also, the seals were subject to considerable wear as they slid along the steel plate, due to the high friction factor of the rubber. On some Corps projects, maintenance and/or replacement of the seals was considered to be too frequent. To remedy this undesirable situation, two things were done:

(1) The side seal rubbing plate was specified to be made of either stainless steel or stainless clad steel.

(2) The ball of the rubber "J" seal was provided with a teflon coating so that the teflon, and not the bare rubber, contacted the side seal rubbing plate. The teflon coating in this application has functioned well and is very durable. It also has a friction factor of about one tenth that of bare rubber. The successful use of these two items makes it highly desirable that all future Corps projects seriously consider using stainless steel side seal rubbing plates with teflon coated rubber "J" seals for spillway tainter gates.

C-4. Streambed Scour Protection

One of the most important features of a navigation dam project is the streambed scour protection that must be provided downstream of the concrete dam structure. Several Corps projects have experienced near-catastrophic situations due to failure of the stone protection to function as intended. Some of the projects had failures during flood conditions, and others had gradual progressive failures over a long period of time. After many years of operation, Dams 52 and 53 on the Ohio River in the Louisville district developed scour holes over 100 ft deep in the streambed downstream of the navigable pass dam sill after the scour protection failed. One section of Dam 52 actually had some downstream movement but did not fail. The overall remedial action required to prevent potential failure of these two dams consisted of filling the scour holes with large stone at a cost of several million dollars.

a. Model tests and studies. Model tests are very helpful in determining the extent and location of scour protection required downstream of a navigation dam structure. In some cases, histories of prototype experiences and required remedial measures are also very helpful in assessing scour protection needs for new projects. Studies of these past experiences have revealed that conservative designs are advisable. Chapter 5, paragraph 5-11 of EM 1110-2-1605 covers the design of downstream streambed protection. See also Chapter 8, Channel Protection, in this manual. Paragraph 8-5 of EM 1110-2-1605 covers rehabilitation methods for failed scour protection in detail, and paragraph 8-6 gives a tabulation of model studies conducted to evaluate major rehabilitations required as a result of scour protection failures at several Corps projects.

b. Other information. Other pertinent information concerning major problems with scour protection at Corps projects is available as follows: Old River Control Structure--Louisiana-New Orleans district; Dams 52 and 53, Ohio River--Louisville district; Red River Dams--Louisiana-Vicksburg district; and Jonesville and Columbia Dams--Louisiana-Vicksburg district. It has also been emphasized by experience and by WES model studies that, in some cases, streambed scour protection is essential during construction to prevent excessive scouring of the streambed material. This is especially true when a steel pile cofferdam is in place and the river flows through a limited opening.

C-5. Cofferdams

a. Steel sheet pile tees and wyes. Welded, in lieu of riveted, steel sheet pile tees and wyes, which were used quite often in the construction of steel pile cells for cofferdams, did not prove suitable because of failures of some welds during driving of these items. The failures were probably due to the fact that the pile material was not of weldable quality, coupled with the high impact on the tees and wyes when they were driven. Subsequently, the Corps took the lead in developing an alternative extruded wye section. However, this extruded wye was used only for a few years and then abandoned. The section was very compact, which resulted in its being very limber and difficult to handle and drive in long lengths. Splicing of the wyes was also not practical. After this, the Corps adopted a policy of using only riveted or high-strength bolted tees and wyes.

b. Use of new and used steel sheet piling. In the early stages of construction, it is advisable to use only new sheet piling because of the considerable risk and liability associated with sheet piling that has inadequate interlock strength or anomalies in the interlocks that cause them to fuse during driving. Use of piling that has been used on two previous projects is subject to approval of higher authority. However, where staged construction is to be used, plans should include reuse of sheet piling in later stage cofferdams, taking the precautions described below.

c. Inspection of used piling. Any steel sheet piling that is to be reused in a subsequent cofferdam should be carefully processed and inspected after it is extracted. The important items in processing and inspecting each sheet of the piling for reuse are as follows: proper handling and storage so that permanent bends are not introduced into the piling and the piling is not damaged to the extent it cannot be reused; visual inspection of the pile webs for any undue rusting, nicks, tears, and splits, and either rejection of the entire sheet or removal of the damaged portion; visual inspection of the interlocks for any damage due to handling, driving, or extraction that would be cause for rejection; gaging of the full length of all pile interlocks with a metal gage to ensure that the

configuration of the interlock is within allowable limits to properly grip the adjoining interlock and develop the proper interlock strength; load testing of coupons from the piling if required to calibrate gaging operations; and reduction of the allowable interlock tension to a conservative value.

C-6. Markland Incident of 1967

This incident resulted from a barge tow breaking loose from an upstream mooring (tied to a tree on the bank that pulled out) and floating into the dam. The barges sank in the dam gate bays with some of the barges wrapping around the piers, preventing the tainter gates from closing during a return from open river conditions. The resulting loss of pool caused major damage to harbors, stranded boats, sloughed highway embankments, and exposed water intakes and sewer outfalls. Figure C-1 shows the removal of one barge from the Markland Dam. Based on this incident, the Corps developed recommendations for future applications as summarized in the following paragraph.

C-7. Summary of Recommendations Based on the Markland Incident

The following edited version of the recommendations contains guidance that relates to the lock and dam design and operation.

  a. Prevention of accidents -- lock and dam.

  (1) Recommend installing remote control systems to provide for operation of spillway gates from the operations building.

  (2) Recommend that Corps of Engineers Regulations prohibit operators from mooring unattended tows within 10 river miles upstream of a dam, except at commercial docks, facilities that have been designed for mooring, government-furnished mooring facilities, or fleeting areas.

Fig. 20 (No number) MARKLAND INCIDENT, 23 May 1967. MONALLO, downstream of Gate 9, attempting to lift sunken barge in Gate Bay 10.

Figure C-1 - Removal of barge wrapped around dam pier - Markland Lock and Dam, Ohio River, 1967

(3) Recommend installation of mooring facilities both upstream and downstream of navigation structures for tows awaiting lockages.

(4) Recommend each district institute training programs for lock and dam personnel to familiarize them with decisions that need to be made in emergency situations.

b. Coordinate efforts with other agencies and navigation interests.

(1) Recommend the Corps of Engineers establish formal liaison and participate with the Coast Guard, other federal agencies, and navigation industry groups in public deliberations and studies concerning the enforcement of safe navigation on the canalized rivers.

(2) Subjects that may be discussed by these and other bodies which could significantly influence the Corps' plans for protection of its navigation structures and on which the Corps' view should be made known include the following: the analysis of requirements for permanent mooring facilities between dams; the development of more specific regulations affecting mooring procedures; regulation of the size and power of tows to ensure safe control of the tow's movement under any reasonably anticipated river conditions; Coast Guard examination and licensing of selected personnel on towboats and self-propelled barges; and mechanical inspection of towboats by the Coast Guard.

c. Engineering modifications. Recommend engineering and economic feasibility studies for installation of protective barriers or lengthened guidewalls upstream of dams, or modification of piers, to prevent major damage to structures by runaway tows.

d. Recovery operations equipment.

(1) Recommend whirler-type derrick-boat of approximately 300-ton capacity be made readily available for emergency use.

(2) Recommend the provision (either by lease or procurement) of a towboat for each district or applicable waterway of sufficient thrust and size to facilitate handling of floating plant that would be used in a recovery operation.

(3) Recommend each district fabricate or procure effective power-driven cutting beams to separate barges wrapped around the dam pier structures.

(4) Recommend technical assistance from HQUSACE and other engineer agencies, such as Engineer Research and Development Laboratories, to determine feasibility of utilizing explosive anchors for emergency mooring of recovery rigging to the lock and dam structures.

(5) Recommend each district examine its capabilities to ensure the following: capability for rigging of heavy wire ropes and chains, and for underwater cutting of steel by torch; development of sounding techniques to accurately determine the underwater positions of sunken barges and obstructions; provision of heavy anchors, and of suitable anchor derrick and winch barge for use in safely positioning floating plant above dam; supply of assorted heavy slings and haul cables with suitable terminal fittings and quick-release devices; development of grappling devices and techniques for quick attachment of haul cables to submerged barges not accessible for conventional attachment; development of equipment and techniques for quick introduction of compressed air into sunken barge compartments; and provision of adequate radio communications during recovery operations between government, navigation, and contractor interests.

e. Modification of lock and dam structures.

(1) Recommend provision of adequate facilities on river walls, piers, and abutments both upstream and downstream of navigation structures for positioning floating plant and for rigging during recovery operations.

(2) Recommend design and procurement of special lifting beams for use with overhead bulkhead cranes.

(3) Recommend engineering, economic, and feasibility studies to provide for use of versatile overhead piggy-back cranes of 50-ton capacity and for clamshell bucket operation and lowering of personnel to work areas.

(4) Recommend engineering and economic feasibility studies for pre-installation of chain slings in gate-bays to expedite removal of objects with bulkhead crane.

(5) Recommend each district install anchor bolts on river wall immediately upstream of the dams to facilitate timely installation of portable winches.

f. Organization.

(1) Recommend each district organize a marine disaster recovery team to ensure adequate supervision of three-shift recovery operations over an extended period.

(2) Recommend each district have a trained, experienced Technical Liaison Office as a single point of contact for coordination of public information activities during emergencies.

(3) Recommend each district maintain a current list of marine contractors and contractors' equipment available for possible use in marine disaster recovery operations.

(4) Recommend that periodic seminars be conducted with key personnel, such as Chiefs of Branches and Construction Resident Engineers, reviewing plans and capabilities and pre-establishing key emergency team members for recovery operations.

g. Applications. Not all the above recommendations will be possible or practical in every Corps district with navigation dams. The recommendations appear to be written more specifically for the navigation dam projects on the Ohio River. Some of the recommendations have been implemented and some have not. It is recognized that some of them would be difficult to design into a project and would be very expensive.

C-8. Maxwell Incident of 1985 (Pittsburgh District)

a. General. On November 5, 1985, as a direct result of storms generated by Gulf storm "Juan," floodwaters in the Monongahela River basin reached record levels in many locations from Charleroi, Pennsylvania, south into the mountains of West Virginia. As a result, as many as 120 barges that had been moored at various landings in the navigation pool broke their moorings and began to float downstream. As they moved in their uncontrolled journey, some were intercepted by towboat crews, some were beached on lowland areas, and others either sank or ended up against highway or railroad bridges or Corps of Engineers dam piers.

(1) At the Maxwell Locks and Dam project located at river mile 61.2 on the Monongahela River, 20 coal barges, both loaded and empty, either individually or in groups of two or three, approached the dam, which had all five of the 84-ft-wide tainter gates in the fully open position. Two barges passed through the gate bays and sank just downstream of the dam. The other 18 impacted on the dam piers and stacked themselves up in positions that required much effort and time to remove. Four empty barges were still afloat and were retrieved by government and contractor towboats. The remaining 14 barges either sank or became entangled against the dam piers or rested broken atop the upper guard wall.

The positioning of the barges around the piers prevented four of the five 84-ft tainter gates of the dam from closing, and this resulted in the eventual loss of the Maxwell pool.

(2) At Locks and Dam 2 at mile 11.2 on the Monongahela River, seven barges (six coal and one tanker) floated uncontrolled into the locks and obstructed navigation through the two lock chambers. The empty tanker barge came to rest nearly perpendicular to the river flow and balanced itself across the land wall just upstream of the land chamber emergency dam. It extended some 80 ft into the upper approach to the large lock chamber. An empty coal barge came to rest atop the upper middle wall and upper guard wall, completely blocking the upper approach to the small lock chamber and virtually all of the approach to the large lock chamber. An empty coal barge remained buoyant on one end just upstream of the river chamber's emergency floodway bulkhead for a short time after the waters receded below the top of the lock walls. As the lock crew removed the last panel of the floodway bulkhead after closing the downstream lock gates, the barge surged downstream, hit the bulkhead panel, and later sank within the small lock chamber. Another empty coal barge sank across the upper middle wall, obstructing both lock approaches. Two more empty coal barges sank across the upstream end of the guard cell. In addition, another empty coal barge impacted against this guard cell and rested atop the two other sunken barges.

(3) Numerous other barges and pleasure boats were observed going over the fixed-crest dam during the height of the flood.

b. Causes of incident.

(1) Highest flood of record on the Monongahela River basin.

(2) Possible inadequate mooring of some barges which broke away and impacted other moored barges, which then also became free-floating and uncontrolled.

c. Major impacts of incident.

(1) The blocking of navigation traffic at Lock 2 and the loss of the Maxwell pool caused navigation traffic to cease on the Monongahela River for some six weeks. As a result of this traffic stoppage and its ripple effect on dependent business interests, plus loss of the barges, tremendous economic losses were incurred.

(2) Structural damage to Lock 2 and to Maxwell Dam.

(3) Damage to highways and highway bridges, and railroads and railroad bridges.

(4) Four municipal water companies with intakes in the Maxwell pool were adversely affected and had to have special help from Corps personnel in order to maintain water services to their customers.

d. Recovery operations. The overall recovery operations to restore normal navigation traffic movement on the river and to return all other affected interests to their fully operational conditions were conducted November 5, 1985, through December 16, 1985. The United States Coast Guard, the commercial towing industry, the affected water companies, salvage contractors, and explosive demolition contractors joined the Corps of Engineers in this recovery effort.

e. Summary of recommendations based on November 1985 Maxwell Incident. The following emergency action plans were suggested by the Pittsburgh district as observations and recommendations for consideration by all Corps organizations when preparing for or responding to similar incidents.

(1) Contingency plans should be developed by every interest that would be affected when a pool is lost.

(2) River recording gages and staff gages should be protected as well as possible from the effects of flooding. Staff gages should be placed in such a way that they can be observed at all times.

(3) Operational contingency plans covering all types of emergencies should be prepared for all district installations, particularly navigation dams.

(4) Minutes of meetings, daily memos of organization activities, and cataloging of slides and photos are necessary during all recovery activities for future report preparation and for use in any subsequent litigation.

(5) Maintain close contacts with the National Weather Service.

(6) Involve affected commercial navigation interests as soon as possible after an incident.

(7) The Corps should take the lead in helping navigation interests develop standardized mooring facilities

and procedures for assuring their proper and continued usage.

(8) Conduct research on physical and economic feasibility of constructing a structural barrier just upstream of each gated dam.

(9) Assure that radio contact will always exist between locks and the District Office.

(10) Determine the availability of horizontal pulling equipment that could be readily contracted in an emergency.

(11) Establish separate account numbers to identify efforts expended on each vessel and the separate identifiable tasks involved in the total operation.

(12) Notify railroad and highway interests when conditions indicate that loss of a navigation pool is imminent.

f. Report. A comprehensive report covering all aspects of this incident can be obtained from the Pittsburgh district. The report, dated December 1986, is entitled "After Action Report - Monongahela River Barge Breakaway Incident - November 1985." Excerpts from the Pittsburgh report have been used in this manual.

C-9. Maxwell Incident of 1990 (Pittsburgh District)

a. General. During December 1989, Pittsburgh district rivers and adjoining streams were frozen with thickening ice. The United States Coast Guard issued three notices to mariners between December 21 and 26, 1989, warning that icing conditions were continuing to worsen along the Allegheny, Ohio, and Monongahela Rivers with reports of ice ranging from four to eight inches thick; that operators of fleeting areas be advised to remain on constant alert for ice floes which might cause barge breakaways when temperature rises occur; and that operators double up on their mooring lines, provide for towboat assistance, and keep a constant surveillance of their fleeting areas to minimize barge breakaways.

(1) A combination of moderation of the weather and heavy rains between December 29, 1989, and January 1, 1990, caused breakup of ice in the river and melting of some snow on the watershed. This combination of events caused flooding and movement of ice on the Monongahela River. The fast-flowing high water and breaking ice jams knocked about 60 barges from their moorings on

January 1, 1990, along the Monongahela and Ohio Rivers and slammed the barges into bridges, locks, and dams. Thirty-seven coal barges moored at a coal-processing facility in the Maxwell pool broke their moorings and began traveling downstream. Upon reaching the Maxwell Locks and Dam, two of the barges passed through the gate bays and sank downstream of the dam, and one barge sank about 1 mile upstream of the dam. The remaining 34 barges collided with the dam piers and stacked up on one another and sank.

(2) Fourteen barges were also adrift in the lower Monongahela River below Maxwell Dam and the adjacent Ohio River. These barges were retrieved before they could cause any extensive damage. Some bridges on the Monongahela and Ohio Rivers were damaged by the runaway barges in the Maxwell pool and downstream to Pittsburgh.

b. Causes of incident.

(1) Flooding and ice floes caused by rising temperatures, heavy rains, snow melt, and ice break-up.

(2) Possible inadequate mooring of some barges, which drifted downstream and caused other barges to break their moorings.

c. Major impacts of incident.

(1) Thirty-four barges collided with the Maxwell Dam piers and sank after piling on top of each other as described above.

(2) One spillway gate at Maxwell Dam could not be closed.

(3) Tainter gates and steel sheet piling at Maxwell Locks and Dam suffered structural damage.

(4) Dollar losses for barges and for coal on barges were sustained.

(5) Five bridges hit by the barges were temporarily closed.

(6) Drawdown of Maxwell pool affected water supply facilities and navigation traffic.

d. Recovery operations. Recovery operations spanned the period January 1, 1990, through February 19, 1990. Restoration of all facilities to pre-incident conditions

involved concentrated efforts by government forces, private towing companies, marine surveyors, salvage contractors, and local affected interests.

e. Summary of recommendations based on January 1990 Maxwell Incident. The following emergency action plans were suggested by the Pittsburgh district as observations and recommendations for consideration by all Corps organizations when preparing for or responding to incidents similar to this one.

(1) Require all facility operators with Waterfront Facility Operation Guides to revise their guides to include precautionary procedures to follow in river icing and ice flow conditions.

(2) Provide a public affairs representative immediately after an incident for media and general public contacts. Station this person in the project manager's office until a separate public affairs facility is established.

(3) Equipment for salvage of sunken barges should include:

(a) Two A-frames with a minimum lifting capacity of 200 tons each.

(b) Four derrick boats with 100 ft of boom and lifting capacities between 50 and 150 tons.

(c) A clam shell bucket without teeth having a capacity of 3 to 4 cu yd.

(d) Two horizontal pulling winches having a minimum pulling capacity of 100 tons each.

(e) Two towboats with a minimum of 800 hp.

(4) When salvage work requires a diver, it is recommended that the Corps require salvage contractors performing diving operations to have a standby diver equipped with scuba gear tend the first diver, due to unpredictable and dangerous conditions associated with the diving activities.

f. Report. A comprehensive report covering all aspects of this incident can be obtained from the Pittsburgh district. The report, dated January 1991, is entitled "After Action Report - Monongahela Barge Breakaway Incident - January 1990." Excerpts from the Pittsburgh report have been used in this manual.