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Closures and Interior Facilities for Levee Projects; Principles, Case Examples, and Risk-based Analysis Concepts

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This report presents the role of closure and interior facilities in the overall performance of levee projects. Hydrologic, physical, institutional, and human considerations that might affect the project reliability during a full range of flood events are examined. Information is compiled and organized to support plan formulation and evaluation within the framework of risk-based analysis. Several Corps of Engineers local flood protection projects located in the Mississippi River Basin are used as case studies illustrate these ideas. Particular attention is given to their performance during the Flood of 1993.

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TABLE OF CONTENTS

Chapter **Page**

Preface ... iii

1 Introduction ... 1
- 1-1. Scope ... 1
- 1-2. Performance, Function, and Workability ... 2
- 1-3. Local Flood Damage Reduction Projects ... 2

2 Interior Flood Damage Reduction Facilities ... 7
- 2-1. Interior Flood Damage Reduction Systems ... 7
- 2-2. Closure Structures ... 9
- 2-3. Ponding Areas ... 14
- 2-4. Gravity Outlets ... 16
- 2-5. Pumping Stations ... 17
- 2-6. Examples of Interior Facilities ... 21
- 2-7. Flood Warning and Project Operations ... 23

3 Case Study: Flood Damage Reduction Projects in the Mississippi and Scioto River Basins ... 29
- 3-1. Local Flood Damage Reduction Projects Reviewed ... 29
- 3-2. The Upper Mississippi River Basin Flood of 1993 ... 29
- 3-3. St. Louis District's Project at St. Louis, Missouri ... 35
- 3-4. Rock Island District's Project at Bettendorf, Iowa ... 44
- 3-5. Rock Island District's Post-Flood Evaluation of Ten Projects ... 48
- 3-6. Huntington District's Project in Columbus, Ohio ... 53

4 Performance, Function, and Workability ... 59
- 4-1. Planning, Design, Maintenance, and Operation Considerations ... 59
- 4-2. Risks Associated with Local Flood Protection Projects ... 60
- 4-3. Risk of Failure Associated with Closure Structures and Interior Facilities ... 63
- 4-4. Issues Identified in the Case Study Reviews ... 66
- 4-5. Consideration of Failure Risk in Planning and Design Studies ... 67

5 Risk-based Analysis Concepts ... 69
- 5-1. Risk and Uncertainty ... 69
- 5-2. Flood Damage Reduction Studies ... 69
- 5-3. Interior Flood Damage Reduction Systems ... 72
- 5-4. Failure Risk for Interior Facilities and Closure Structures ... 74
- 5-5. Failure to Discharge Interior Flows ... 74
- 5-6. Failure to Block River Flows ... 79
- 5-7. Considerations in Developing Closure Risk Estimates ... 86
- 5-8. Concluding Remarks ... 90

TABLE OF CONTENTS (continued)

Chapter **Page**

References 91

LIST OF TABLES

Page

2-1 Local Flood Protection Projects 22
3-1 Case Study Local Flood Protection Projects 30
3-2 Levees Constructed or Improved by the USACE in the Upper Mississippi River Basin 34
3-3 Closures for Columbus Project 55
3-4 Rate of Rise During Historic Storms Scioto River at Columbus, Ohio 57

LIST OF FIGURES

2-1 Spatial Configuration of a Levee Project 7
2-2 Cross-Section of a Typical Interior System 9
2-3 Stoplog Structure 11
2-4 Swing Gate 12
2-5 Miter Gate 13
2-6 Rolling Gate 14
2-7 Trolley Gate 15
2-8 Rainfall Hyetograph and Stage Hydrograph for a Flood Event 25
3-1 Location of Local Flood Protection Projects 31
3-2 Upper Mississippi River Basin 32
3-3 Areas Flooded in 1993 33
3-4 Local Flood Protection Project in St. Louis 37
3-5 Local Flood Protection Project in Bettendorf 45
3-6 Local Protection Project in Columbus 54
5-1 Three Basic Functions and Derived Probability-Damage Function 70
5-2 Uncertainty Reflected in Basic Relationships 71
5-3 Interior Stage-Probability Relationship 76
5-4 Probability Distribution Function Representing Interior Stage Uncertainty 78
5-5 Exterior and Interior Stage Relationships 82
5-6 Example Interior Stage-Exceedance Probability Function 84
5-7 Frequency-Stage Curves for Closure Risk 87

Preface

This document is a product of the U.S. Army Corps of Engineers Civil Works Research and Development program. It documents the results of investigations into closures and other ancillary facilities associated with the 'interior areas' that are part of levee protection projects. Interior areas refer to the location where local runoff is blocked by levees and for which 'interior facilities' must be provided to ensure proper functioning of the project. Closures are a particularly important feature of interior facilities in that they ensure that flood waters do not inadvertently spill onto the protected flood plain. The material documented herein records performance aspects of closure and other ancillary interior facilities important in the formulation and evaluation of levee projects. The material herein draws from Corps levee project case studies, Corps technical manuals, recent technical reports on the Mississippi Flood of 1993, and the professional literature. It also overviews emerging concepts in the risk-based analysis area as applied to closures and other ancillary interior protection features. This document was prepared by Dr. Ralph Wurbs while serving an Intergovernmental Personnel Act appointment at HEC and made extensive use of case studies prepared by several field offices under the guidance of Dan Barcellos, HEC staff member. The project was supervised by Mike Burnham, Chief, Planning Analysis Division, HEC with consultation and advice from Darryl Davis, Director, HEC.

Chapter 1
Introduction

1-1. Scope

Performance, function, and workability are fundamental considerations in planning and design of local flood damage reduction projects. Expanded risk-based analysis methodologies have recently been developed and applied. This report addresses these concepts as applied to (1) closure facilities for openings in levees and floodwalls, and (2) components of interior flood damage reduction systems associated with levees and floodwalls. (For purposes of brevity, the term "levee" will be used hereafter to imply floodwalls as well as levees.) The objectives of the report are to:

- outline factors and considerations that affect the performance, function, and workability of closure structures and interior flood damage reduction systems, and
- organize available information to facilitate incorporation of risk and uncertainty in the formulation and evaluation of these project components.

Providing openings through levees and floodwalls for highways, railroads, and pedestrian walkways is often less expensive than ramping over or routing around the levee or floodwall. Closure facilities are then required to block the openings during floods. The reliability or the risk that closures will not occur as planned during a flood must be considered in the plan formulation and evaluation process.

Interior flood damage reduction systems typically include gravity outlets, pumping stations, pump discharge outlets, collection facilities, pressurized storm sewers, and detention storage or ponding. They are designed to pass the interior runoff through or over the levee. Performance of the local flood protection project includes the proper functioning of these components. Uncertainties are also inherent in essentially all aspects of predicting the performance of system components for the full range of floods, including floods that exceed system capacity. Recognition and consideration of relevant uncertainties are required throughout the process of planning, design, implementation, and operation of the project.

This report presents the role of closure and interior facilities in the overall performance of levee projects. Hydrologic, physical, institutional, and human considerations that might affect the project reliability during a full range of flood events are examined. Information is compiled and organized to support plan formulation and evaluation within the framework of risk-based analysis. Several Corps of Engineers local flood protection projects located in the Mississippi River Basin are used as case studies illustrate these ideas. Particular attention is given to their performance during the Flood of 1993.

1-2. Performance, Function, and Workability

EC 1110-2-280 (U.S. Army Corps of Engineers 1994c) states the Corps policy to fully consider performance, function, and workability throughout the project development and implementation process. The expected performance must be clearly defined and well documented to insure that function and workability considerations are adequately incorporated into the project plan to support the project purposes. Performance, function, and workability are defined as follows.

Performance refers to the ability of a project to accomplish its planned purpose of reducing flood damage by reducing flooding or removing or modifying the damage susceptibility from the flood threat. Performance includes consideration of residual flooding from the full range of flood events, including floods that exceed the project capacity.

Function refers to the way physical component features of a project contribute to its overall performance. A variety of components, such as gates, pumps, closure structures, and flood warning systems, must individually and collectively function properly.

Workability addresses the responsibilities of organizations and people in implementing, maintaining, and operating the project. Institutional, organizational, legal, financial, and community responsibilities and actions are considerations in assuring that a plan is workable and will support the expected project performance objective.

1-3. Local Flood Damage Reduction Projects

Precipitation and resultant streamflows are highly variable and subject to extremes of floods and droughts. Rivers naturally overflow their channels periodically and inundate adjacent floodplain lands. Floodplains are natural components of river systems, providing additional storage and conveyance during periods of high flow. Floodplain lands along rivers and streams provide many advantages for cities, agriculture, and related human activity. Both urban and agricultural development commonly occur on land subject to some degree of flooding.

a. Flood Damage Reduction Measures. People adopt various strategies for dealing with the flood threat. Flood damage reduction plans and actions typically involve combinations of structural and nonstructural measures.

Structural measures reduce damages by partially controlling flood waters. Reservoirs, diversions, and watershed management practices reduce flow rates by storing the flood waters, changing the flow path, or reducing the volume, respectively (Hydrologic Engineering Center 1990). Channel projects allow a given flow rate to be conveyed at a lower stage. A levee blocks the river from overflowing into the floodplain.

Nonstructural measures reduce the damage susceptibility or redistribute the costs associated with flood damage. Floodplain management actions involving development regulations encourages prudent use of floodplain lands. Flood proofing reduces the susceptibility of individual properties to flood damage by raising or protecting individual structures. Flood warning-preparedness programs reduce damage by facilitating emergency actions during flood events. Emergency relief and recovery assistance transfer flooding cost from the individual property owner to the community or nation.

The National Flood Insurance Program (NFIP) has been a driving force in implementation of nonstructural measures in communities throughout the nation since the early 1970's. The NFIP was established by the National Flood Insurance Act of 1968, significantly modified by the National Flood Disaster Protection Act of 1973, and amended by several other legislative acts. The program is administered by the Federal Insurance Administration (FIA) of the Federal Emergency Management Agency (FEMA). Although participation by local communities is voluntary, of 21,926 communities identified by the FIA as flood prone, 18,023 or 82% had joined the program as of September 1990 (Federal Interagency Floodplain Management Task Force 1992). The NFIP encompasses a broad range of floodplain management activities. Key elements of the NFIP include (1) making flood insurance available for existing properties and (2) encouraging and supporting local communities in their regulation of floodplain development.

Federal activities in constructing flood control improvements nationwide date back to the Flood Control Act of 1936. Prior to 1936, Corps of Engineers flood control activities were focused primarily in the lower Mississippi River Basin. Over 300 of the dams and reservoirs owned by the Corps of Engineers include flood control as a primary project purpose. The Corps of Engineers is also responsible for operating the flood control pools of a number of Bureau of Reclamation reservoirs. Flood control is a primary operational purpose of the multiple-purpose Tennessee Valley Authority system. The Natural Resource Conservation Service (former Soil Conservation Service) has constructed about 6,000 flood control dams under its Small Watersheds Program.

b. Levees. Levees are the most common type of flood damage reduction works (Federal Interagency Floodplain Management Task Force 1992). Levees were probably the first structures built for flood control by European immigrants to North America. The first levee in the Mississippi Valley was constructed at New Orleans in 1717, and levees have been built and rebuilt along the Mississippi River ever since. An estimated 25,000 miles of levees have been built nationwide. The Corps of Engineers has designed and constructed about 10,500 miles of levees, most of which are operated and maintained by local agency sponsors (National Research Council 1982).

FEMA has established minimum design, operation, and maintenance standards for levees for making special flood hazard area determinations. Areas behind levees that meet specified standards are shown on NFIP maps as areas of moderate flood hazard. Approximately 1,000 communities nationwide, or about 5.5% of the communities identified as flood prone, have one or more levees credited on NFIP maps. These levees have a total length of about 9,000 miles and protect about 5,000 square miles of land (Federal Interagency Floodplain Management Task Force 1992).

Levees are longitudinal barriers constructed on one or both sides of a river. Earthen levees are generally more economical to construct than concrete or steel floodwalls. Floodwalls are often advantageous compared to levees in urban areas with limited availability of land. Levee systems normally run parallel to a river with a tie in to high ground at the upstream and downstream ends. In some cases, a ring levee will encircle the protected area.

Deferred maintenance and rehabilitation for all types of infrastructure is a major concern nationwide. There is no information on the condition and safety of levees and floodwalls that covers all levels of government and classes of ownership (Schilling 1987). However, it is known that a large proportion of private or locally constructed levees are poorly designed and maintained. Some privately built levees have been constructed without regard to design standards at all. Corps of Engineers levees are designed and constructed following stringent criteria and standards. However, inadequate nonfederal maintenance, due to funding and other constraints, may be a problem at projects constructed by the Corps as well as at nonfederal projects. According to the Federal Interagency Floodplain Management Task Force (1992), levee overtopping or geotechnical failure is involved in approximately one-third of all flood disasters.

c. Levee Protection. Levees provide only partial protection for reasons associated with overtopping, structural or geotechnical failures, closure failures, or interior flooding. Levees are designed to exclude flood waters up to the capacity exceedance stage. If properly constructed and maintained overtopping by a larger flood does not imply a design failure. The consequences of these capacity exceedance events must be considered. Even without overtopping, a risk exists of levee breaching due to structural or geotechnical failure and or a closure failure. Local interior runoff may be a major flooding problem. Blocking of flow by the levee contributes to interior flooding. Seepage from the river under or through the levee may add to the flood water to be handled by the interior flood damage reduction system. Surfacing ground water also may contribute to the problem.

Areas behind levees and floodwalls may be subject to greater risk of flood damage than the protected population expects or understands. The expectation of flood protection may motivate the populous to increase floodplain development and into thinking they are totally protected from all future floods. Long periods between major floods may result in a lapse in maintenance and preparedness. A levee breach or a failure of a closure could occur, unexpectedly without warning, resulting in a rapid rise in interior flooding levels. After a breach, the downstream portion of a levee system may act as a dam, causing prolonged interior inundation of considerable depth.

d. Institutional Setting for Federal Projects. Local flood protection projects designed and constructed by the Corps of Engineers are turned over to local sponsors for operation and maintenance. The Flood Control Act of 1936 stipulated what became known as the "a-b-c" requirements of local cooperation, that local interests should:

(a) provide without cost to the United States all lands, easements, and rights-of-way necessary for construction of the project;
(b) hold and save the United States free from damages due to the construction works; and
(c) maintain and operate all the work after completion in accordance with regulations prescribed by the Secretary of the Army.

The Flood Control Act of 1941 modified the 1936 Flood Control Act to apply the a-b-c requirements only for local flood protection projects and not for flood control reservoirs. The hold and save requirement (b) was modified by the Water Resources Development Act of 1974 to not include damages due to the fault and negligence of the United States or its contractors.

Cost sharing provisions for federal water projects were modified by the Water Resources Development Act of 1986. For single purpose structural flood control projects, local project sponsors are still required to provide lands, easements, and rights-of-way and to perform relocations. However, local interests must also pay 5%, in cash, of the project first costs assigned to flood control. The local sponsor's share of the cost for project construction must fall within the range of 25% to 50% of the total federal/nonfederal cost. Either the nonfederal or federal share is increased as necessary to have the nonfederal costs within the 25-50% limits.

Prior to the Water Resources Development Act of 1986, costs for feasibility studies and advanced engineering and design for Corps' studies and projects were borne totally by the federal government. Under current cost sharing policy, after an initial reconnaissance study at federal expense, 50% of feasibility study and design costs are the responsibility of nonfederal sponsors.

The Corps of Engineers normally conducts feasibility studies in response to Congressional authorization as expressed by either resolutions of pertinent committees or public laws. Construction projects are generally authorized by the Congress through omnibus legislation based on the results of feasibility studies. Advanced engineering and design follows Congressional authorization of a construction project. Alternatively, small local flood protection projects have been studied and implemented under the continuing authority provided by Section 205 of the Flood Control Act of 1948. Under this program, specific Congressional authorization is not required for individual projects.

Chapter 2
Interior Flood Damage Reduction Facilities

This chapter focuses on two features of levee projects:

- closure facilities for the openings in levees that are provided for highways, railroads, and pedestrian walkways, and
- interior flood damage reduction facilities for discharging interior flows over or through levees.

These facilities are described along with a discussion of flood monitoring, forecasting, and warning systems used to support the operation of closure structures and interior facilities.

2-1. Interior Flood Damage Reduction Systems

Runoff from interior watersheds must be passed over, through, or around levees. Figure 2-1 illustrates the spatial configuration of a local flood protection project. The location of the system of levees is called the line of protection. The line of protection ties into high ground on either end. The levee protects the interior area encompassed by the line of protection and high ground. Stormwater runoff from this area and additional higher interior watershed areas must somehow be discharged into the river. Other sources that contribute to interior flows include: seepage from the river under or through the levee; wave overtopping; leakage or spills through closure structures; groundwater; combined sanitary and storm sewer flows; and municipal and industrial wastewater treatment plant effluent.

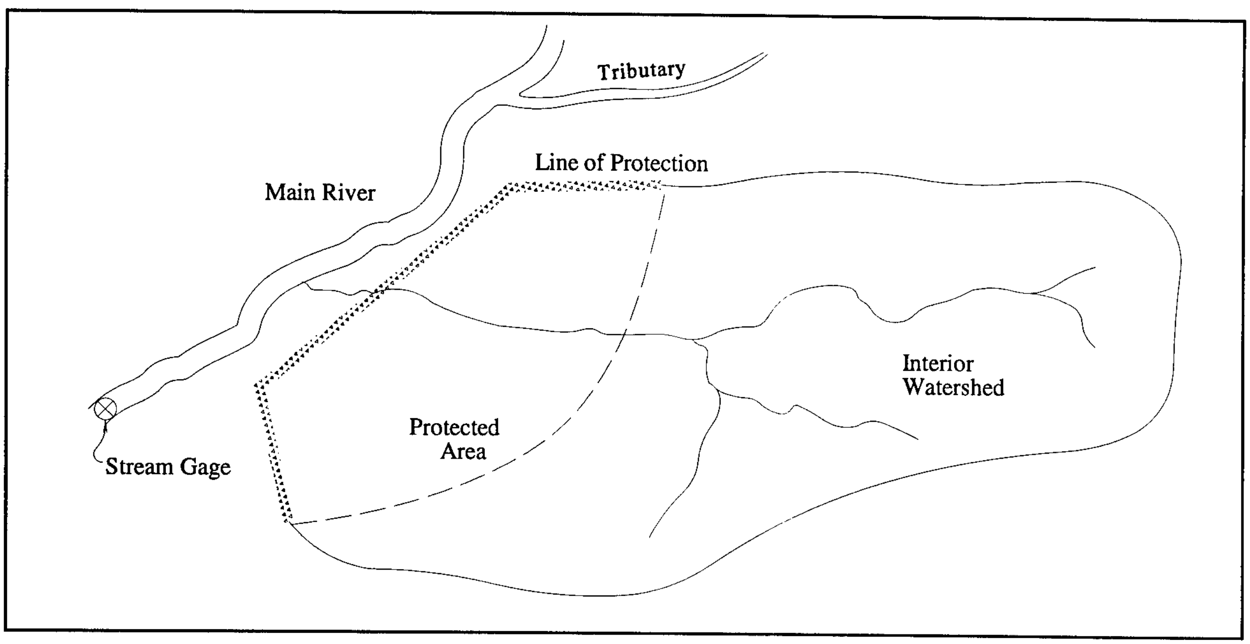


Figure 2-1. Spatial Configuration of a Levee Project

Facilities for dealing with storm runoff and other flows in the watersheds behind levees include:

- flood damage reduction facilities located at the line of protection used to discharge flows to the river;
- diversion facilities that divert flows to locations outside of the watershed, without crossing the line of protection;
- flood damage reduction measures located at remote sites throughout the interior watershed that are not directly connected to the levee;
- stormwater management and drainage facilities located throughout the interior watershed;
- wastewater collection, treatment, and discharge facilities; and
- combined sanitary and stormwater sewers.

This document focuses on the first category of measures listed above, which deal with discharging interior flows over or through the levee to the river. Ponding areas, gravity outlets, and pumping stations are discussed in Sections 2-3, 2-4, and 2-5, respectively. These measures are components of the overall system for managing excess flows in interior watersheds.

a. Interior Systems. In general, flooding, and stormwater management concerns in interior watersheds of levee projects are essentially the same as in similar watersheds for which there are no levees. The differences are related to dealing with the levee blocking natural flow paths to the river. Flood damage reduction measures applied elsewhere are also adopted for interior watersheds. Such measures include regulation of floodplain land use, emergency preparedness programs, floodproofing, channel improvements, detention storage facilities, storm sewers, combined sanitary and storm sewers, and surface drainage systems. Other levees may also be located along streams in the interior watersheds of levee projects.

In some cases, flows from interior watersheds are diverted around levees rather than crossing the line of protection. Diversions are one of the basic types of measures adopted for local flood protection projects. Diversion projects may or may not also involve levees. For a levee project, a diversion could affect either external river flows or interior flows. Exterior river flows may be diverted upstream to bypass a proposed levee, allowing the levee height to be decreased. Interior flows may also be diverted through conduits or open channels to enter the river downstream or upstream of the line of protection. Diversion facilities include a control structure to divert all or a portion of the flow from a stream or sewer system, conduits or channels for conveying the diverted flow, and an outlet structure where the flow is discharged into the river or another tributary stream.

b. Measures at the Line of Protection. Interceptor channels and conduits, detention ponding areas, gravity outlets, pressure sewers, and pumping stations are located at or near the levee. These facilities intercept, store, and discharge the interior water into the river. To reduce the number of outlets through the line of protection, interceptor channels or sewers convey discharges from multiple collector subsystems to a single outlet or pumping station.

As long as the interior water level is higher than the exterior water level, the interior flows are passed through gravity outlets. Outlets include gates and valves to prevent backflow from the river from entering the interior during high river stage. Outlet conduits may convey either free surface or pressure flow. Pressure conduits have inlets located so that the flow entering the conduit is higher in elevation than the maximum river elevation. A pressure conduit may convey flows to and through a levee that enter the conduit some distance from the levee.

When the river stage exceeds the interior stage, interior flood water is stored and/or pumped over or through the levee. In some cases, a significant amount of storage capacity may be provided for temporary detention or ponding. In other cases, pumping capacity may be sized to handle the inflow at the site with little or no storage.

The following sections describe those facilities located at or near the line of protection that are used to discharge the interior flows over or through the levee to the river. These measures include detention ponding areas, gravity outlets, and pumping stations, which are illustrated in Figure 2-2.

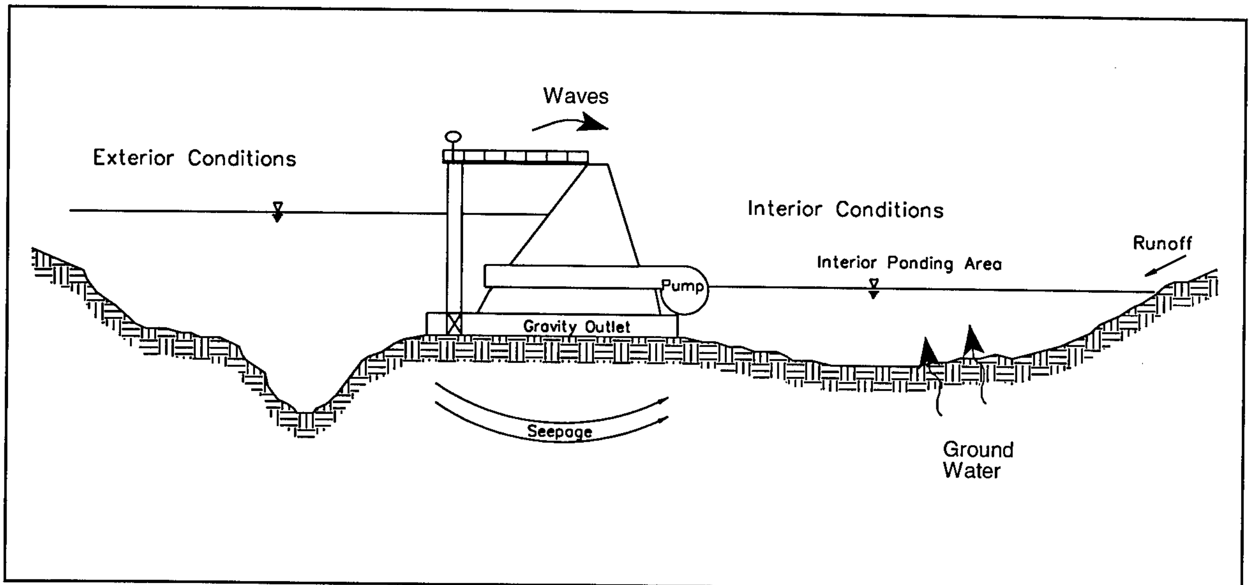


Figure 2-2. Cross-Section of a Typical Interior System

2-2. Closure Structures

Normal transportation activities involve people and vehicles traveling over, through, or around levees. The alternatives of raising highways, railroads, and walkways to ramp over the levee or rerouting them around the levee are advantageous in that there are no openings to deal with during floods. However, construction costs and other factors often result in openings being selected rather than ramping or rerouting alternatives.

A flood damage reduction project may include one or many openings in levees, for highways, railroads, and pedestrian walkways, that must be closed during floods. All of the projects described in Chapter 3 have at least one closure structure. The St. Louis and Columbus projects have 38 and 14 closures, respectively. Opening widths of 20 to 40 feet are common for highways and railroads but sometimes exceed 100 feet. The Bettendorf project has an opening that is 220 feet wide. Openings for pedestrian walkways are usually less than 20 feet wide. Opening heights may range from a foot to greater than 30 feet.

The case study projects discussed in Chapter 3 illustrate a variety of levee and closure structure configurations and designs. The structural design criteria provided by EM 1110-2-2705 (U.S. Army Corps of Engineers 1994a) were developed based on a review and evaluation of closure structures constructed throughout the Corps. The two primary types of closure structures are stoplogs and gates. There are several types of gate facilities. Sandbags also are commonly used during flood emergencies either supplemental to or in lieu of stoplogs or gates. Stoplog closure structures are usually less expensive than gates. Gates typically can be closed quicker with less effort. Sandbags cost less than stoplog or gate structures but require more time and effort to place. Examples of other types of closures include the panels used for large openings at the St. Louis project and the folding wall used for the wide opening at Bettendorf.

a. Stoplog Closure Structures. Stoplog closure structures typically consist of sets of aluminum or steel beams or logs stacked horizontally across the opening as illustrated in Figure 2-3, which is reproduced from EM 1105-2-2705 (U.S. Army Corps of Engineers 1994a). For narrow openings, one set of logs may span between support slots at the edges of the opening. For wider openings, intermediate removable support posts are required as shown in Figure 2-3. Aluminum stoplogs are lighter, but steel has the advantage of greater strength for the same dimensions. Lifting equipment is often required for large openings. Sandbags, plastic sheeting, and other available means are typically used to reduce leakage through a stoplog closure. In some cases, portable pumps may be used to return leakage to the river. Storage facilities are required for the stoplogs, removable posts, and accessories. When secured areas are available, closure items may be stored on uncovered concrete slabs or pedestals. Otherwise, a storage building is required.

Advantages of stoplogs relative to gated closure structures include:

- Fabrication methods are simple and economical.
- Initial cost is usually less than for gate structures.
- Stoplogs are easily placed for narrow and low openings.

Disadvantages include:

- Installation of stoplogs requires more time than closing gated structures.
- For openings with intermediate support posts, additional time is typically required to clean the post sockets during installation.
- Dependable and timely flood warning is necessary, since a relatively long lead time is required to mobilize personnel and equipment for installation.
- Special lifting equipment is usually required for installation of stoplogs in wide openings.
- A secured area or storage building is required to prevent damage by vandalism or loss by theft.
- Quality planning and personnel training at regular intervals are required.

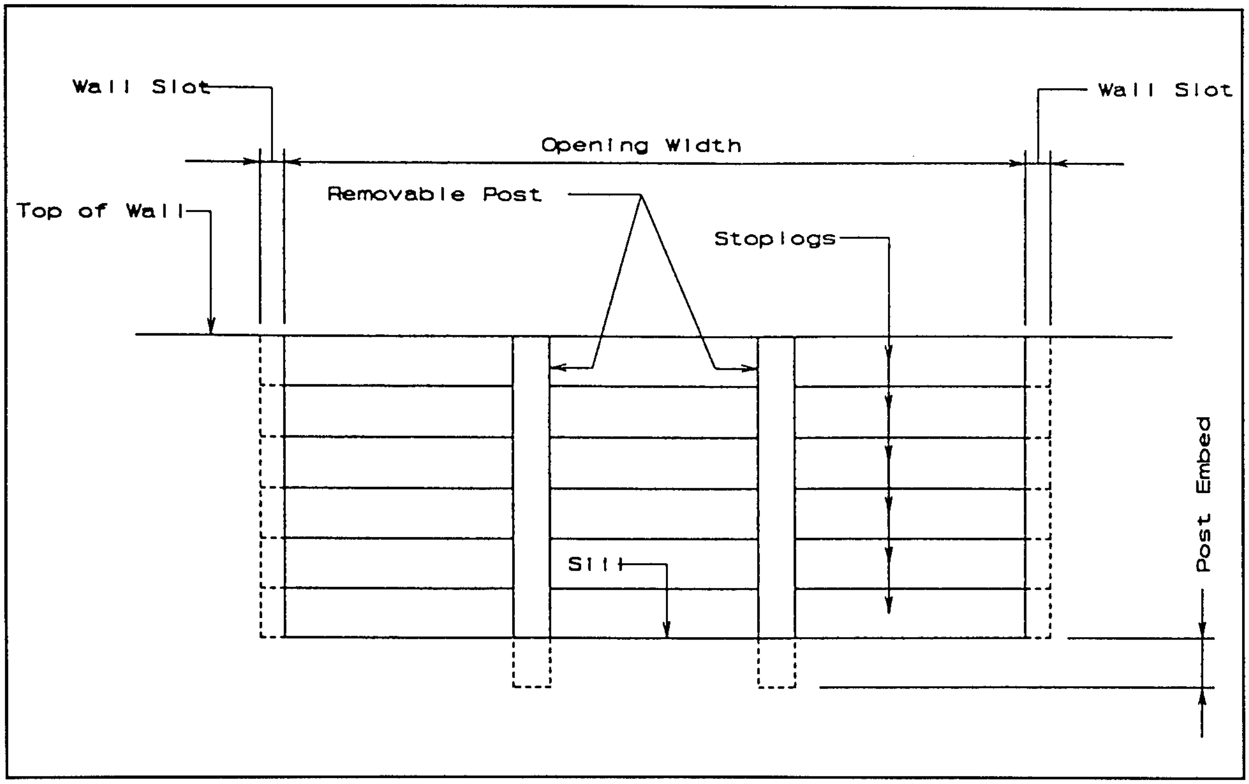


Figure 2-3. Stoplog Structure (U.S. Army Corps of Engineers 1994a)

b. Gate Closure Structures. The most common types of gates used for closure structures are swing, miter, rolling, and trolley gates. Drawings of typical gates representing each of these types are provided as Figures 2-4, 2-5, 2-6, and 2-7, which are reproduced from EM 1110-2-2705.

As illustrated in Figure 2-4, swing gates are composed of two or more horizontal girders, vertical intercostals, vertical end diagrams, a skin plate, and diagonal braces. Swing gates are supported on one side by top and bottom hinges attached to a support structure. One end of the diagonal linkage rods is permanently attached to the free end of the gate leaf, and the other end is attached to the support structure when the gate is closed. For openings widths of up to about 40 feet, a single gate leaf is typical. Double gate leafs are used for wide openings. Double gate leafs may be stabilized by a removable center post or tie back linkages. Rubber J-seals are attached to gates to form a continuous water-tight seal between the gates and supporting walls and sill of the opening. Closure facilities may include winches or motor vehicles to accomplish closure during strong winds.

As illustrated in Figure 2-5, miter gates consist of two leaves that form a three-hinged arch when the gates are in the closed position. The gate leaves are attached to support piers by top and bottom hinges. Each gate leaf is composed of horizontal girders, vertical intercostals, vertical end diaphragms, a skin plate, and adjustable diagonal tension rods. The diagonal tensioning rods are required to prevent twisting of the gate leaves due to their dead load and must be properly tensioned after the gates are installed so that the gates hang plumb and miter properly. Support structures for miter gates are usually more costly and difficult to design than for other types of gates because deflections must be minimized to allow the gates to miter properly. J-seal assemblies are provided for water tightness. Winches or motor vehicles are used to close the gates during strong winds.

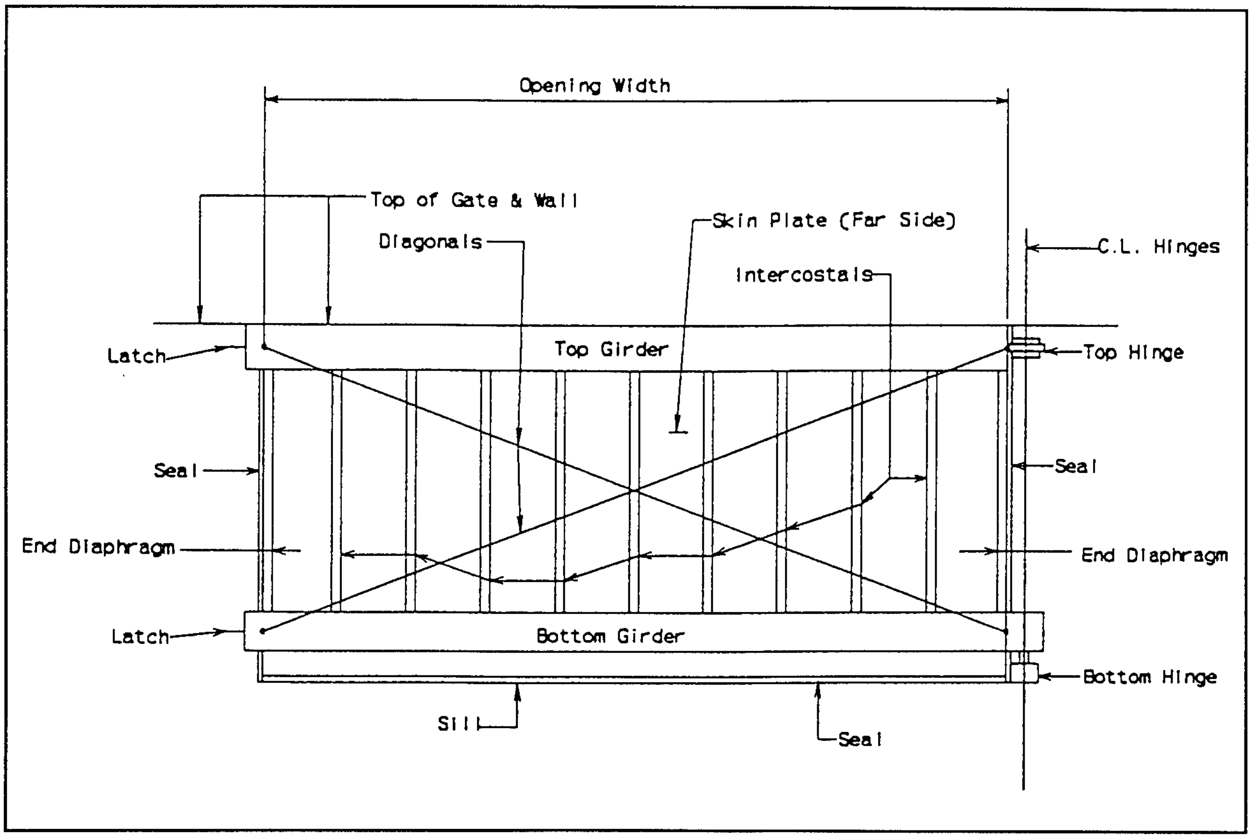


Figure 2-4. Swing Gate (U.S. Army Corps of Engineers 1994a)

As illustrated by Figure 2-6, rolling gates are supported by wheels that roll on tracks embedded in the sill across the closure opening and storage area. A winch may be mounted at the site for gate operations. Alternatively, the gates may be moved by a cable attached to a truck motorized winch or pulled directly by a truck. Gates along fast rising streams may be designed to be closed and opened from the protected side of the levee. Rolling gates are composed of a structural steel frame covered by a water barrier skin plate. J-seals are attached to the ends and bottom of the gates to form a water-tight seal. For opening widths of up to about 30 feet, a single line of wheels and stabilizing trolleys are often sufficient. Gates with two lines of wheels are adaptable to wider openings.

As illustrated by Figure 2-7, trolley gates are suspended from trolleys running on an overhead rail and beam supported by the floodwall. The gates are opened and closed by a winch arrangement similar to that used for rolling gates. Likewise, trolley gates are usually composed of top and bottom horizontal girders, other secondary framing members, and a skin plate.

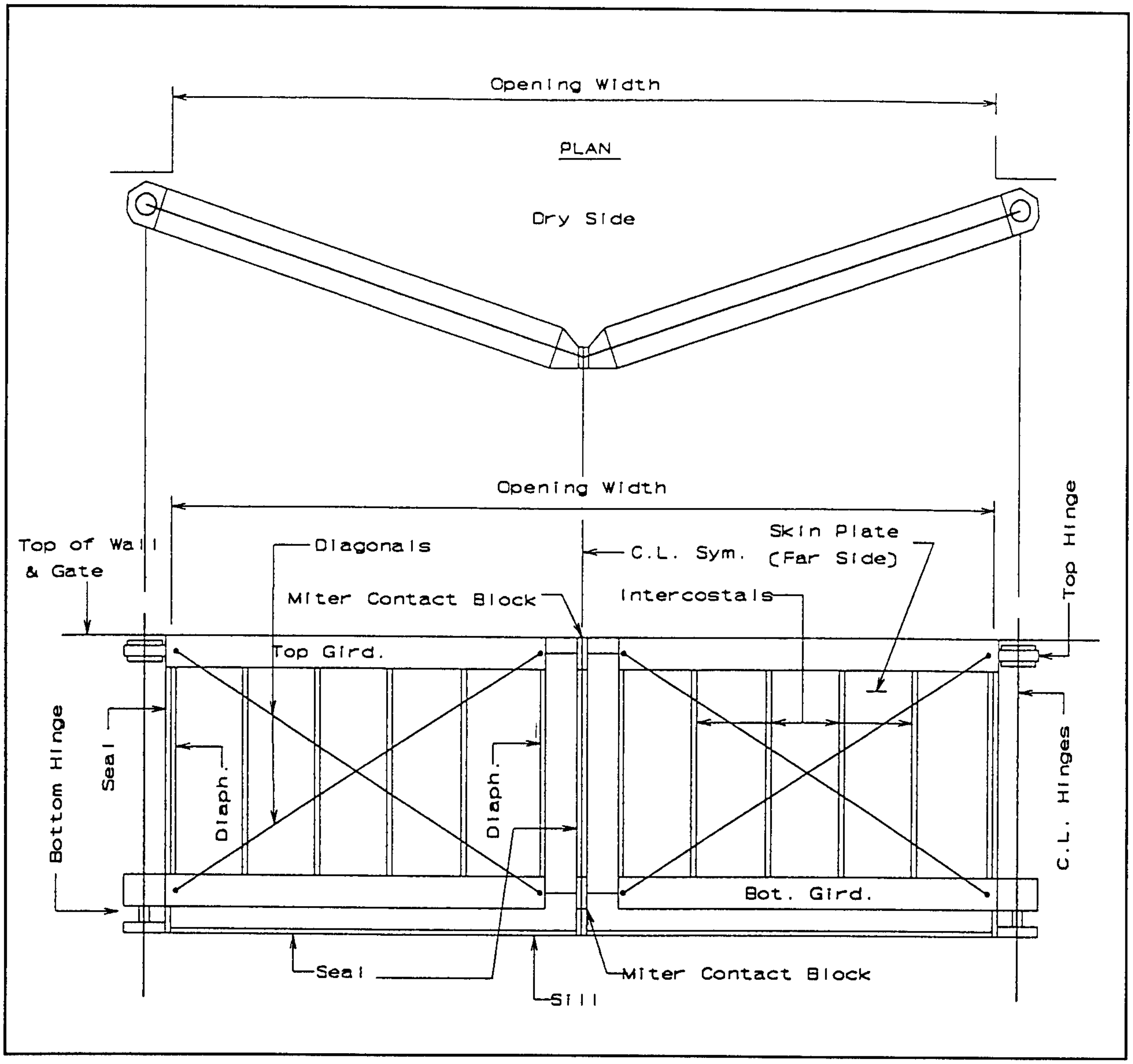


Figure 2-5. Miter Gate (U.S. Army Corps of Engineers 1994a)

Advantages and disadvantages associated with gated closure facilities include the following considerations.

- Closures can be made quickly by unskilled personnel with proper instruction.
- Closing most gates requires some type of equipment which must be brought to the site.
- Gates are typically more expensive to construct than stoplog facilities.
- A level storage area adjacent to the opening is required to store the gate in an open position. In most cases, storage buildings are not required. Intermediate support posts for double leaf swing gates are an example of an exception in which storage is required.
- Different gate designs are limited to various opening widths. Rolling gates may be designed for essentially any opening width.
- Nonlevel sill surfaces are a problem requiring special attention.

- Gates may be difficult to operate during high winds, requiring special equipment.
- Trolley gates may be rendered inoperative due to permanent overhead support members being damaged by vehicles or other sources, or removable overhead support members or their anchorages being damaged during removal or placement operations.

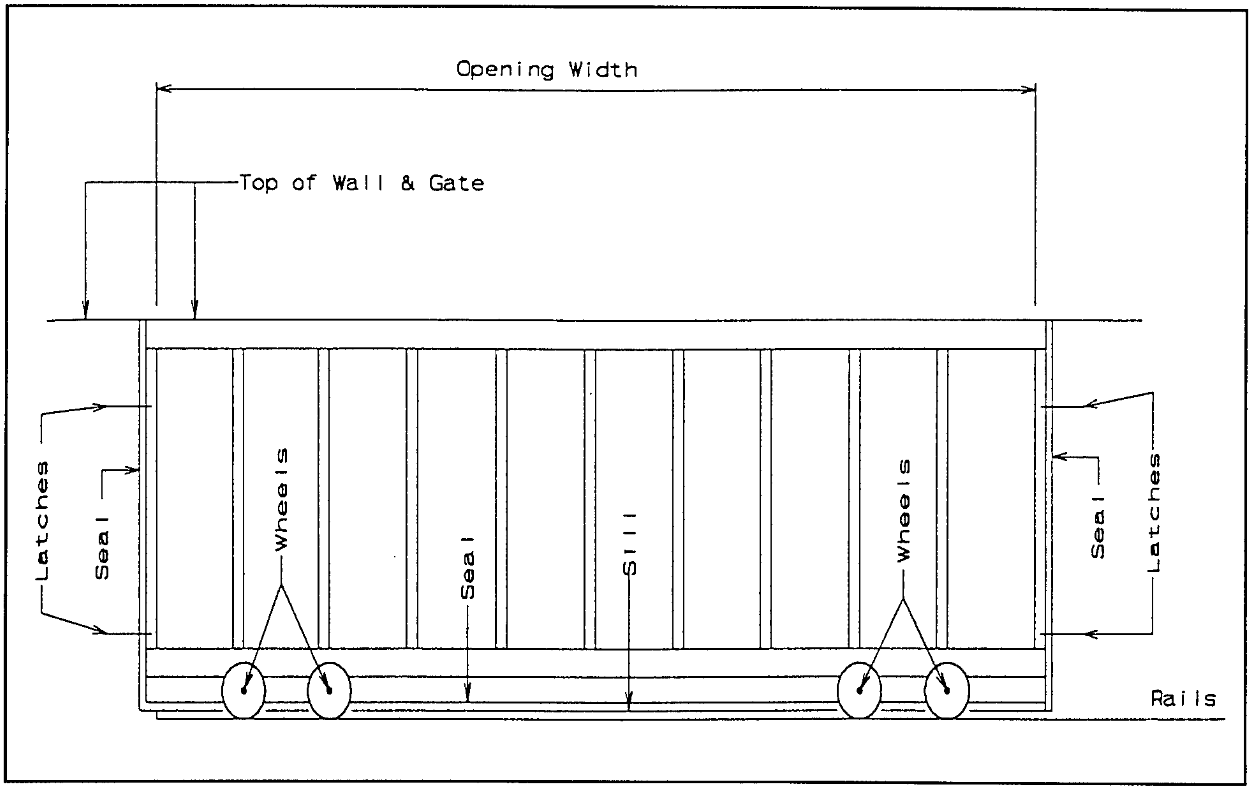


Figure 2-6. Rolling Gate (U.S. Army Corps of Engineers 1994a)

2-3. Ponding Areas

Ponding or temporary storage of water occurs whenever inflow rates exceed outflow capacities. For a specified level of flood protection, there is a tradeoff between the required storage volume and the outlet flow capacity. Storage capabilities allow a reduction in outflow discharge capacity. Thus, the size and costs of gravity outlets and pumping stations may be decreased. Detention storage also increases the reliability of flood protection by providing additional time for gate and pump operations before damaging water levels are reached.

A ponding area may consist of natural or excavated basins and/or vacant lots, parks, and streets and lowlying developed areas. A local flood protection project may or may not include significant designed ponding areas with detention storage depending on availability of suitable sites. Ponding areas are normally located adjacent to the gravity outlet or pumping station, but may be remote from these facilities, connected by channels or conduits. Topography, flow patterns, safety, and land use as well as siting of other project components govern the location of detention storage. The future availability of ponding areas and associated storage must be considered as part of the planning and design process. Land use controls, and sound legal right

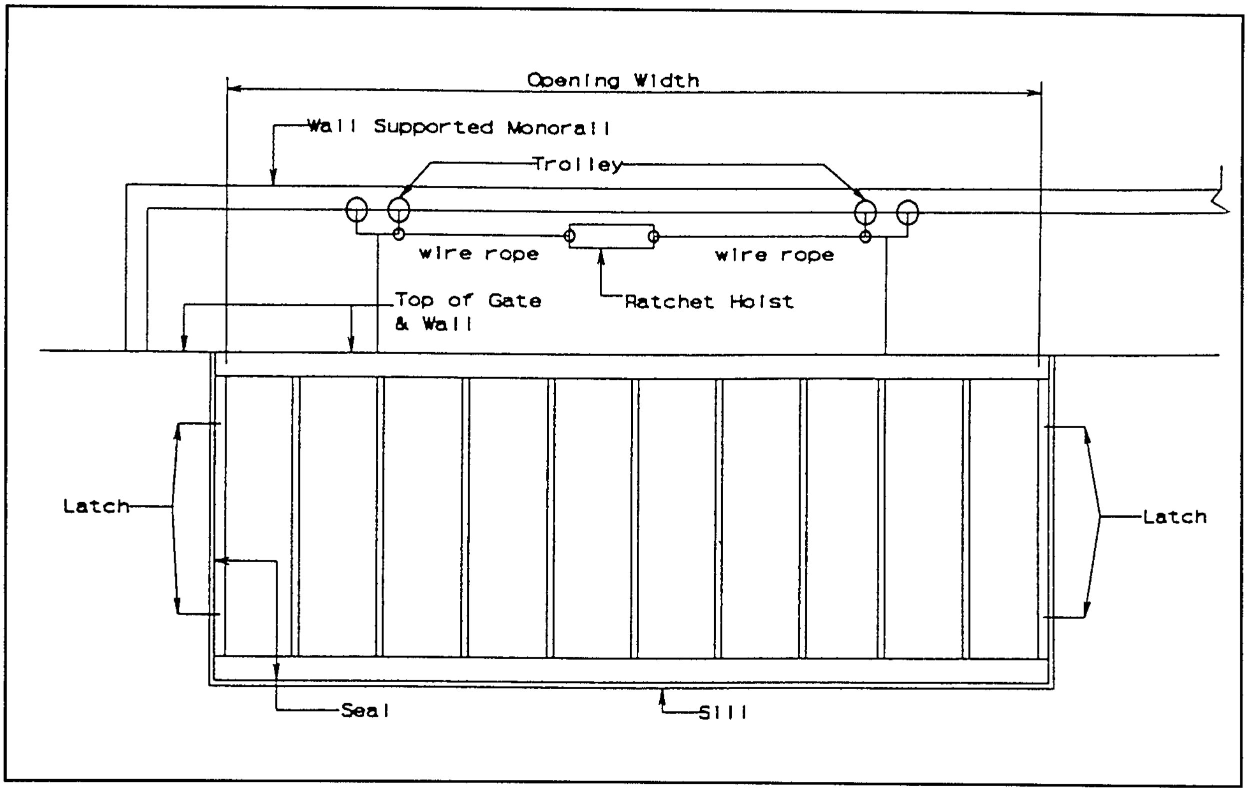


Figure 2-7. Trolley Gate (U.S. Army Corps of Engineers 1994a)

are essential to prevent encroachment into ponding areas by development. Allowance for sediment accumulation, plant growth, and future development are included in the sizing analyses of the storage area for future conditions. Aesthetically attractive and environmentally sensitive ponding areas may be designed for use as parks and open spaces during nonflood periods. For convenience when describing ponding areas, to assist in selecting appropriate land use controls, and to communicate expected project performance ponding areas may be provided into classes. Ponding area classes are generally described a s follows:

- Primary or designated ponding areas are locations where accumulation of interior runoff is expected to pond at frequent intervals in sufficient volumes to warrant dedication of the area primarily for use as a ponding area. Such areas often consist of low-lying lands that are unimproved or where improvements are not seriously damaged by temporary ponding. However, primary ponding areas may sometimes include sections subject to appreciable adverse effects from occasional ponding. Primary ponding areas are preserved by acquisition in fee or by flowage easements where fee acquisition is impracticable.
- Secondary ponding areas normally include sparsely developed land, streets, parks and other areas that are expected to be utilized occasionally for temporary ponding with minor to moderate damages. Flowage easements, building codes, and other legal regulations normally govern land use and construction practices in secondary ponding areas.
- Extreme flood ponding areas may be subject to infrequent flooding from interior runoff, with damages ranging from moderate to critically severe. Flowage easements are seldom practicable for such areas, but building codes and other regulations may be used to minimize potential damages from possible infrequent flooding.

2-4. Gravity Outlets

Gravity outlets are culverts or conduits through the line of protection that allow discharge of interior flows into the river without pumping. Conduits may flow full under pressure or partially full as free surface flow. The relationship between interior and exterior water levels is a critical factor in operating gravity outlets. Gravity outlets are typically located at or near where the line of protection intersects the natural or existing conveyance system or detention area. The required outlet flow capacity may be reduced by detention storage. An outfall channel conveys flows from the outlet to the river. Gates are required in gravity outlets to prevent backflows during high river stage. The term *service gates* refers to gates used for normal operations. Emergency gates are used when service gates fail to function properly. Various types and configurations of gates are in use.

a. Service Gates. Common types of service gates include automatic flap-type gates, hand-operated or motor-operated slide gates, or a slide gate with a flap attachment. In typical installations, flap gates are installed on the river end of the gravity outlet conduit, with appropriate headwall, wingwall, and outlet floor structure to provide protection against river currents, debris, sediment, or ice accumulations insofar as practical. In some cases, flap gates are installed in gatewells on the river side of a levee or floodwall. Slide gates are typically located in gatewells on the river side of the levee, with a foot bridge or other provisions for access during floods. Slide gates are also installed in gatewells that are an integral part of a concrete floodwall or pumping station. In some cases, slide gates with flap gate attachments have been installed in gatewells on the river side of levees or at the river end of outlet conduits.

Slide gate operation is more reliable than flap gates. Emergency closures also can be made readily with slide gates if obstructions prevent gate closure. For example, the gatewell could be filled with sandbags as an emergency closure. Slide gates are more expensive than flap gates, and personnel are required at the site to close and open gates at the proper times. Automatic flap-type gates are advantageous for rivers that rise suddenly or under circumstances where the river stage is likely to fluctuate within a short height above or below the normal gate-closing stage for protracted periods of time during flood seasons. Use of a flap-type gate as the primary service gate usually requires provision of a supplemental emergency gate or other special provisions for emergency closure. Slide gates often have been considered to provide sufficient safety to eliminate the need for a supplemental emergency gate or other closure structure.

b. Emergency Closure Facilities. Service gates may fail to close completely during critical flood periods due to clogging by debris, mechanical problems, or other causes. The resulting flow from the river may create major problems for the interior flood protection system and/or inundate otherwise protected areas. The purpose of emergency gates and other forms of emergency closure of gravity outlets is to minimize these risks. The likelihood and consequences of gate failure vary between types of gates and the circumstances at particular local flood protection projects.

Provisions for emergency closures include supplemental gates in gatewells or structural features at the ends of the conduit to facilitate placement of temporary closures. Structural provisions at the river end of the conduit may include stoplog slots or metal hooks or eye-bolts placed in the concrete headwall structure to simplify the attachment of commercially built gates or improvised closures. Ends of conduits should be finished in a smooth and regular form to facilitate obtaining a reasonably tight seal by emergency installations. A concrete apron extending a few feet from the end of a conduit provides a base for placement of sandbags during an emergency. In situations where floating craft may be used for placement of emergency closures in relatively deep water, piling projecting above the design flood profile is desirable for locating the outlet and installing the emergency closure.

*c. **Operation of Gravity Outlets.*** Gravity outlets are regulated using fixed or variable gate closing and opening elevations. Other projects, especially agricultural levees on large slow rising rivers, have generally used a fixed gate closing/opening operating plan. When the driver rises to about the gravity drain invert the gates are closed and not opened until the river falls below this elevation. Such a plan has the advantages of being simple, using a minimum of manpower, any debris in a gate can be removed while the river is low, and only basic flood forecast information is needed. It has the obvious disadvantage of not optimizing gravity outflow which may result is desirably high ponding levels. To optimize gravity discharge and minimize interior ponding levels, many urban levee projects use a variable gate operating plan. Gates are kept open any time there is adequate head, pond level higher than the river, for discharge by gravity. This operating plan requires trained operators to be a t the gravity outlet, or close by, during flood operations, possibly motorized gates, and accurate real time data on the ponding level and river stage at each gravity outlet. Operators can obtain real time flood data by the simple means of reading staff gages located a the drain on the interior and exterior side of the levee. A variable gravity drain operating plan cost more than a fixed elevation plan but provides more benefits.

2-5. Pumping Stations

Pumps provide capabilities for discharging interior flows over or through the levee even when high river stages block gravity outlets. Pumping plays a major role in flood damage reduction for projects with limited ponding capacity and where exterior and interior flooding are highly dependent such that there is a high likelihood of blocked gravity outlets coincident with interior flooding. At the other extreme, some levee projects require no pumps, with only detention storage and gravity outlets needed for dealing with interior runoff. Evaluation of pumping station justification is part of the planning process.

The feasibility of pumping stations is based on economic, risk, and other considerations. When pumping stations are more costly than other features, they are generally considered as a last added element during the plan formulation process. Pumping stations are one of the more vulnerable features of a flood control project. Consequently, the reliability of each component of the pumping plant and its overall performance are primary considerations in project planning, design, maintenance, and operation. Guidelines for designing pumping stations are provided by EM 1110-2-3102, EM 1110-2-3104, and EM 1110-2-3105 (U.S. Army Corps of Engineers 1995a, 1989b, 1962).

*a. **Station Layout.*** Pumping stations are normally located adjacent to the line of protection. Generally, a larger capacity station is preferable over several smaller ones. The station should be aligned to allow as direct inflow patterns as possible from the inlet channel, storm sewer, or ponding area. Asymmetrical flow into pump bays causes problems with circulation, uneven velocity distribution, vortices, and generally poor pump performance. Adjacent gravity flow structures can be located in an offset position and still perform adequately. Pumping stations are normally located at the landside toe of earthen levees. Construction of the pumping station integral with a concrete floodwall will, in general, minimize the hazard of discharge line failure. Due to economic or operational advantages, pumping stations are sometimes located at more hazardous locations on the river side of a levee or floodwall. Access and protection of the equipment during floods are essential for stations located on either side of a levee.

Pumping stations normally pump stormwater runoff, seepage, wave overtopping and other sources. In some cases, pumping stations handle combined sewers with both stormwater and sanitary sewage and/or industrial wastewater. Protection against corrosive fumes and vapors is a much greater problem in handling wastewater than stormwater. Combined flow stations should provide sufficient baseflow capacity for peak sanitary and industrial wastewater flows and runoff due to light rains.

b. Pumps. Flood water pumping stations are normally of the wet-pit (sump) type employing vertical mixed-flow or axial-flow pumps. Submersible pumps are also sometimes used. Provision is typically made for maintaining the sump in a dry condition during inoperative periods. Flood waters are usually pumped directly from detention ponds, ditches, or storm sewers. When wastewater is combined with stormwater, separate smaller submergible nonclog pumps are often provided for dry weather base flows.

The total pump capacity is determined based upon design river stages, interior ponding stages, and inflow rates. The design capacity can be provided by alternative combinations of number and sizes of pumps, with more small pumps or fewer large pumps. The number and resulting size of stormwater pumps are determined based on economics while also considering the risk and consequences of pump failure. The greater the number of pumps, the smaller the reduction of the total station capacity if one pump malfunctions. Generally, reliability is increased with multiple pumps, but the lowest cost is obtained with a minimum number of pumps. EM 1110-2-3102 recommends a minimum of two pumps. Additional pumps may be warranted, to reduce the impacts of a pump malfunctioning, in urban situations where a pump failure could cause significant property damage and life threatening conditions. A decision to divide the total station capacity between more pumps for reliability reasons must be justified and documented.

EM 1110-2-3102 (U.S. Army Corps of Engineers 1995a) provides the following guidelines regarding standby pumping capacity to be followed in selecting the number and size of pumps.

- For pumping stations where seepage flows are more than 30% of the total required capacity, standby capacity of 100% should be available for the failure of any pump.
- For stations pumping stormwater only or combined flows of stormwater and sanitary sewage, normally no standby capacity should be provided.

*c. **Control of Pumps.*** Selection of the type of control for a levee/floodwall pumping station is based on providing maximum reliability consistent with economical design. In most cases, controls providing for manual start and automatic stop are preferable. Manual start has an advantage of being able to start pumping at a low stage if a large inflow is forecast or at a higher stage if the inflow will not contribute to flooding damage. However, automatic start and stop controls may be advantageous for projects where limited sump capacity and inflow conditions make manual starting impractical due to short operating cycles, or where economy is obtained by using pumps of different sizes operating in a predetermined sequence. Disadvantages of automatic controls include increased complexity due to additional control equipment, greater cost, and reduction in reliability. Automatic controls are more susceptible to deterioration than manual controls and require more frequent inspection and maintenance. Automatic controls must have a manual backup. Although exceptions are possible for projects with automated or remote control features, Corps policy basically requires that competent operators be on duty at pumping plants whenever the necessity for pump operation appears to be imminent.

*d. **Power Supply.*** The reliability of a pumping station is highly dependent on the reliability of its electrical power supply. The power supply and electrical equipment for pumping stations should be selected on the basis of reliability under emergency conditions. Corps of Engineers policy is to not provide additional emergency or standby power supply facilities unless the power supply is considered unreliable. For federal projects, the local sponsor is responsible for the supply of electric energy after a project becomes operational. Corps responsibilities include provision of electrical equipment and facilities, including the extension of existing power transmission facilities required to make the power available at the pumping station site. Power supply features of a pumping station construction project may vary from a simple overhead service drop at utilization voltage to extensive installations involving transmission lines, switching, and transformer equipment.

The reliability of a power source depends upon the number, type, size, and location of generating facilities and interconnections with other power systems. Factors affecting the reliability of the supply connection between the power source and the pumping station include the length, location, and type of construction of the connection, and the characteristics of the switching equipment between the connection and supply circuits. Evaluation of the adequacy of power sources includes consideration of the maximum power available, capacities of pertinent transmission and distribution lines and substations, voltage regulation characteristics, the power company's maximum permissible in-rush current limitations and short circuit characteristics, and capabilities for directing power distribution under emergency conditions.

*e. **Sumps.*** The water enters a pump directly from a sump which is supplied from a ponding area or directly by a sewer or channel. For stations pumping from ponding areas, the design water level in the sump is set by the maximum permissible ponding elevation, above which adjacent properties are flooded and damages occur. For stations pumping directly from sewers, the water level in the sump depends upon elevations at which damages occur in the protected area, the hydraulic gradient in the sewer system, and the condition of the sewers. The station-operating floor should be at least a foot above the maximum sump water surface elevation. During nonflooding periods when the pumps are not used, sumps are normally dewatered by gravity drainage or sump pumps.

During the early 1990's, the USACE Waterways Experiment Station developed the formed suction inlet that provides improved hydraulic performance over that sometimes experienced with the rectangular wet-pit sump. Optimal flow conditions occur when the water approaches the pump with as uniform a velocity as possible and with minimum disturbance from the flow toward other pumps in the station.

Surges in sumps are a consideration in pumping station design and operation. Surges are rapid changes in pressures and water levels caused by sudden changes in discharge rates in pipelines flowing full. Pump operation involves quick changes in flow rates that may generate surge problems. The pressure waves may damage pipes and equipment and cause drastic depth increases in sumps and other openings. The height to which water will surge in a pump sump is a function of the characteristics of the piping system connecting to the sump, the sump volume, and the change in pump discharge rate. Certain features of pumping stations and sewer systems coincidentally provide a dampening effect on surges. Sewer laterals, manholes, pump sumps, gatewells, and trash rack wells act as surge tanks to dissipate the pressure wave. Surge tanks or surge basins may be provided specifically for handling surges caused by pump operation.

Trash racks (bar screens) are normally used to screen all flows into flood protection pumping stations before reaching the pumps. Otherwise, trash and debris will clog and damage the pumps. Trash racks are designed to allow incoming flows to pass through the rack before reaching any pump intake, flow to be evenly distributed over the submerged rack surface, and raking to be accomplished coincident with pump operation. Either mechanical or hand-raking equipment is used to clean debris from the racks for stations handling only flood flows. Mechanical rakes are used for sanitary and combined sewer flows.

f. Pumping Plant Discharge Facilities. Pipes and appurtenant fittings and structures are required to convey the discharge from the pumping plant to the river. Normally, each pump has a separate discharge pipeline. For small capacity pumping stations, two pumps may be connected to the same discharge pipe. Pumping water under pressure through conduits through or under levees should be avoided due to the possibility of leakage and/or piping (seepage along outside of pipe) causing damage or breaching of the levee. Pressure conduits through the levee must be properly designed with adequate strength, flexibility, restraint to axial movement, and seepage protection. Pumping stations may be located in floodwalls, with the discharge through the riverside wall of the station into a gravity conduit or open discharge chamber.

Due to the potential for levee settlement, discharge conduits over levees normally should be limited to metal pipes, preferably ductile iron or coated steel, suitable for use with flexible couplings. Concrete conduits are not suitable. The invert elevation of the highest point of the discharge line should be the same as the top of levee at the pumping station site. Pipe may be supported on the surface of the levee and should be completely covered by mounting except on the river side of the levee where the pipe should be placed in a trench to avoid concentration of levee erosion by flood flows. Covering the pipe facilitates levee maintenance and provides access over the pipe for pedestrian and vehicular traffic.

Discharge pipes should include both normal operation and emergency backup means of preventing backflow from the river when the pumps are not operating. For discharge lines over a levee or floodwall, the invert of the highest point is normally at or above the design protection level. An emergency means should still be provided for stopping backflow if the river stage exceeds the project design capacity. If the invert of the highest point along the pipe is below the design protection river stage, a valve should be installed at the highest point. For discharge pipes under or through levees and floodwalls, both service valves and separate emergency closure provisions are required.

Outlet structures are required at the end of the pipes to protect the levee from erosion from the pump discharge and river current. The effect of erosion from the discharge is minimized when pumps are in operation with the river stage is above the discharge outlet. Outlet structures normally consist of a concrete headwall, wingwalls, apron, and cutoff wall. Riprap protection is typically provided for the bottom and sides of discharge channels.

2-6. Examples of Interior Facilities

The case study local flood protection projects described in Chapter 3 illustrate various configurations of facilities for handling interior flood waters. The projects are listed in Table 2-1 with pertinent information regarding interior facilities.

The St. Louis project is the largest of the 12 local flood protection projects. The upstream and downstream reaches of levee/floodwall in St. Louis are separated by high ground and protect areas of 2,530 and 630 acres, respectively. Both reaches protect long, narrow strips of commercial and industrial land along the river front, with the distance from the line of protection to high ground varying from a few hundred feet to 2,000 feet. The total drainage area behind the two levee reaches is 16,000 and 7,418 acres, respectively, for a total interior drainage area of 23,418 acres. The project includes 28 pumping stations with capacities varying from 20,000 to 1,800,000 gpm. The pumping capacities of the 28 stations average about 180,000 gpm. The project includes 44 gravity outlets ranging in size from 15-inch diameter conduits to a 23-feet horseshoe-shaped tunnel. There are essentially no ponding areas. The interior area is completely sewered. The flows from the sewer systems are discharged through the gravity outlets and pumping stations without detention storage. The interior is drained by 44 separate sewer systems, which existed when the federal project was constructed. The project included about 11 miles of replacement sewers, ranging in diameter from 8 inches to 20 feet.

The federal project in Des Moines, authorized in 1944 and constructed in the late 1960's, included no pumping facilities. In the feasibility studies, pumping plants were found to not be economically justified. At that time, interior runoff was considered to be a local responsibility. Motivated by operational experiences during the 1993 flood and previous floods, the City has constructed 16 pumping stations. Five more pumping stations are under construction, and three others are being designed. The stations are generally 2 or 3 pump installations, using submersible pumps in wetwells. The City is also purchasing additional trailor-mounted portable pumps, with power units.

Table 2-1
Local Flood Protection Projects

City	Interior Drainage Area (acres)	Primary Ponding Area (acres)	Number of Pumping Stations	Pump Capacity (gpm)	Flood Warning System
Columbus, Ohio	1,166	none	2	100,000 & 180,000	yes
St. Louis, Missouri	23,418	none	28	20,000 - 1,800,000	no
Upstream Reach	(16,000)			(mean = 180,000)	
Downstream Reach	(7,418)				
Bettendorf, Iowa	1,126	7	2	40,000 & 150,000	yes
East Moline, Illinois	6,492	60	1	63,000	no
Milan, Illinois	2,210	55	2	5,000 & 150,000	yes
East Milan	(767)	(25)	(1)	(5,000)	
West Milan	(1,443)	(30)	(1)	(10,000)	
Rock Island, Illinois	5,160	none	1	100,000	no
Muscatine, Iowa	172	8	1	24,000	yes
Burlington, Iowa	325	9	1	5,000	no
Des Moines, Iowa	8,996	none	see note	-	no
Ottumwa, Iowa	926	none	1	9,000	no
South Quincy, Illinois	10,000	10	1	86,000	no
Hannibal, Missouri	64	64	1	40,000	no

Note: The local flood protection project in Des Moines was originally constructed with no pumping stations. The City has since added 16 pumping stations, with 8 more planned or under construction.

The Bettendorf flood protection project, authorized in 1968 and constructed in 1982-1987, includes seven gravity outlets with gatewells and two pumping stations with capacities of 40,000 and 150,000 gpm. Four of the seven gravity outlets are located adjacent to the smaller pump station. These 6 feet wide by 4 feet high conduits have gatewells with motor operated slide gates. The gravity outlet gates are closed, and the pumping station pumps are turned on whenever the Mississippi River reaches specified levels. One other gravity outlet is located elsewhere in the main levee, and the remaining two are in a tie-off levee along a tributary stream. The smaller pumping station has three identical 24-inch pumps, with 24-inch diameter discharge pipes that run over the levee and discharge into a gatewell on a gravity outlet. The pumps receive water from a pond located adjacent to the pumping station that, in turn, is feed by two remote ponds. The other larger pumping station is located above an interceptor sewer that collects from several feeder storm sewer systems. The pumping plant draws water directly from the interceptor sewer. During high river stage, five identical 36-inch pumps discharge directly to the river through 34-inch diameter lines fitted with flap gates. There are no gravity outlets or ponding area adjacent to this pumping plant. However, during normal low river stages, the interceptor sewer drains by gravity to the river at a location further downstream. At both pumping stations, after the pumps are initially started manually, they are then operated by automatic preset float control devices. At both stations, when the pumps are not in operation, inlet gates isolate the pump chambers from the inflow sources, and sump pumps keep the chambers dry.

As indicated by Table 2-1, two of the other local flood protection projects also include two pumping stations, and all the remaining projects each have one pumping station. Four of the 12 projects have essentially no ponding areas. The pumping stations, gravity outlets, and other features of the interior systems for the projects are discussed in Chapter 3.

2-7. Flood Warning and Project Operations

Timely and accurate information regarding flood stages is required for effective operation of closure structures and interior flood damage reduction facilities. The functional performance of these project components is dependent on the effectiveness of flood monitoring, forecasting, and warning systems. Warning time is required to mobilize personnel, operate gates and pumps, and place stoplogs and/or sandbags. Although closure of traffic openings to prevent flooding is the critical concern, operations personnel are also interested in optimal timing for both closing and reopening closures from the perspective of minimizing disruption of transportation to the extent feasible, consistent with public safety. Efficient operation of gravity outlets and pumping stations allowing ponding depths and durations to be minimized. Flood monitoring, forecasting, and warning are important for reservoir operations, emergency evacuations, and other aspects of flood emergency response activities as well.

For the several case study local flood protection projects reviewed in Chapter 3, the only occurrence of severe property damage during the 1993 flood was as a result of insufficient time being available to close an opening. The closure failure at the Des Moines project was for a railroad opening through the levee along the Raccoon River. The sandbag closure was not made due to lack of warning time, and flood waters spilled into the city. This example illustrates the importance of warning and response times in considering project reliability and appropriate closure structures.

a. Warning Time. Flood characteristics vary tremendously between regions and watersheds. On the one extreme, for the Mississippi River at St. Louis, officials may anticipate that flood stage will likely be exceeded, for one or two weeks before the flood stage is actually reached. However, even with the Mississippi River, as noted in Chapter 3, needs exist for improvements in the accuracy and timeliness of flood forecasts. On the other extreme, for steep impervious smaller watersheds, peak stages may occur within hours of the beginning of an intense storm event. For example, 140 people died during the July 30, 1976 flash flood in the Big Thompson Canyon in the Rocky Mountains of Colorado. That evening, an intense storm resulted in 11 inches of rain falling in a short time over a small area of the steep rocky watershed of the Big Thompson River. A flood wave swept through the canyon within hours catching people with almost no warning.

For several of the projects discussed in Chapter 3, the levees provide protection against floods on smaller tributaries characterized by more flashy floods as well as against floods on the slow rising Mississippi. Inadequate flood monitoring, forecasting, and warning capabilities associated with the tributaries were noted as significant problems, and the Mississippi River was also a concern in this regard. For many other local flood protection projects throughout the country, streams rise much quicker with less warning time than those discussed in Chapter 3. Rates of rise and other flood characteristics can also be highly variable between different floods at the same location.

Future events are always uncertain. River stages at a location, during the next several hours and days into the future, are predicted based upon:

- the current river stage and rate of rise at that location
- gaged flow hydrographs at other upstream locations in the basin
- rainfall measured at gages in the watershed
- in some cases, information regarding snow accumulation and melt.

Weather forecasts may also be used to predict future rainfall amounts. However, precipitation forecasting is highly uncertain.

The river flow response to a storm depends upon the (1) amount and spatial and temporal distribution of the rainfall, (2) rainfall-runoff response characteristics of the watershed, (3) antecedent soil moisture conditions of the watershed, (4) hydraulic characteristics of the river system, (5) antecedent reservoir storage contents and operating policies, and (6) operation of other types of water control structures. Watershed models are used to predict the runoff response to specified precipitation and to route flows through the river system. Widely used watershed models include the Hydrologic Engineering Center's *HEC-1 Flood Hydrograph Package*; the USACE North Pacific Division's *Streamflow Synthesis and Reservoir Regulation (SSARR)*; and the National Weather Services' *Antecedent Precipitation Index (API) Model* and *Sacramento Soil Moisture Accounting Model.*

Warning time is not a precisely defined measure. This concept is illustrated by the stage hydrograph and rainfall hyetograph shown in Figure 2-8. For this particular flood, the river level reached flood stage at a particular point in time. The problem is to predict beforehand if and when the flood stage will be exceeded. Dotson and Peters (1990) note the inverse relationship between warning time and reliability. It is desirable to know as soon as possible if and when the future flood stage will occur, but accuracy or reliability is also important. False alarms are to be avoided. There is a tradeoff between timing and accuracy. Initiation of rainfall is an indication that a flood is a possibility. However, most rain storms do not result in the river reaching flood stage. Early predictions are often highly unreliable. As the event continues and the river rises, the prediction becomes more reliable.

The timing of the actual occurrence of precipitation and streamflow is represented by Figure 2-8. Watershed precipitation-runoff modeling allows estimated streamflows to be predicted before they actually occur. Observations of flows at upstream locations also allow prediction of future flows further downstream. Warning time is reduced by time requirements for (1) observing or measuring precipitation and or streamflow, (2) communicating field data to forecast personnel, (3) performing the modeling and analysis tasks involved in developing and interpreting forecasts, and (4) communicating information to operations personnel.

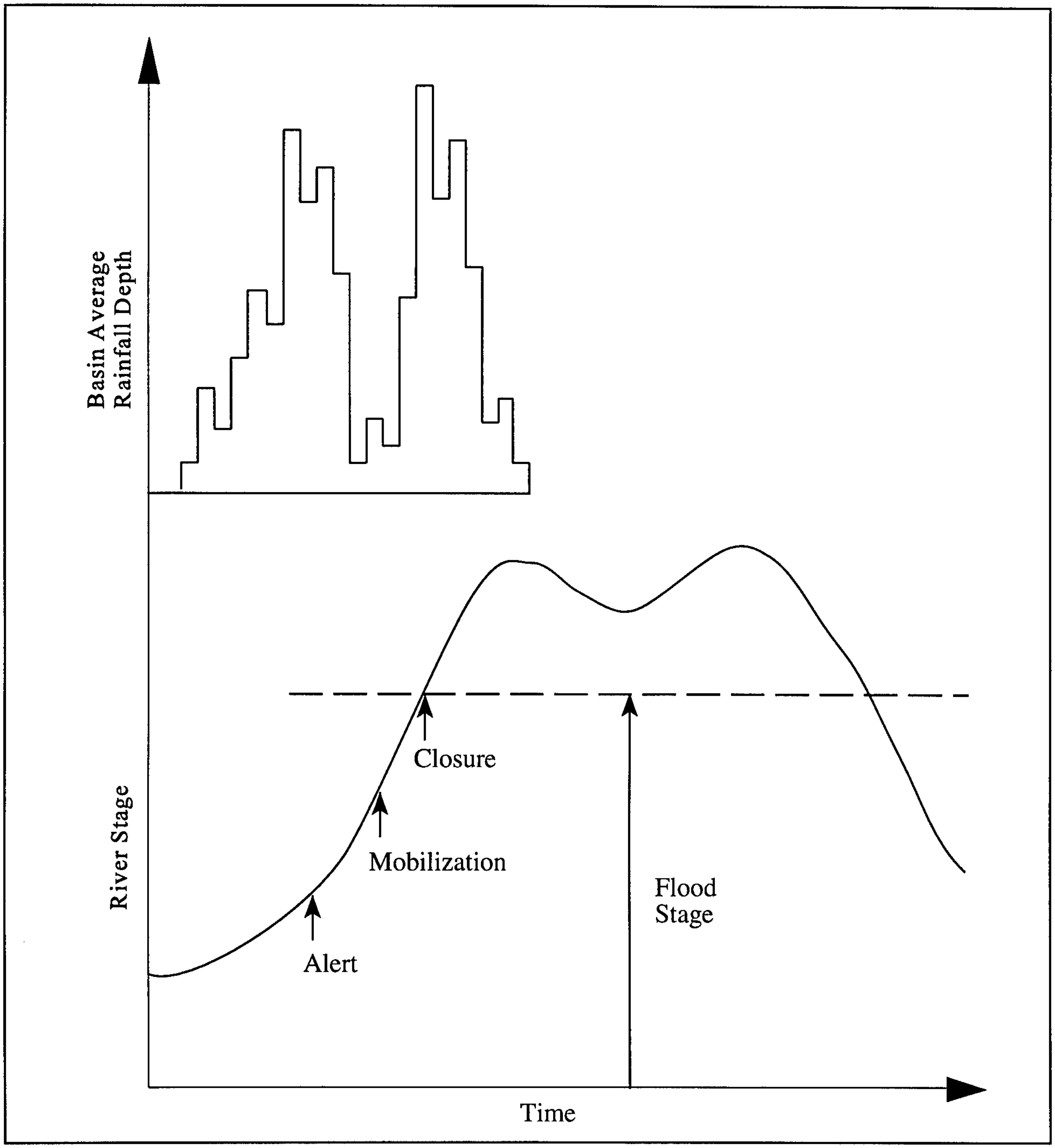


Figure 2-8. Rainfall Hyetograph and Stage Hydrograph for a Flood Event

The hydrologic warning time characteristics of a particular watershed and local flood protection project are studied by analyzing historic gaged floods and synthetic floods developed using watershed precipitation-runoff models.

b. Flood Forecasting and Warning Systems. The National Weather Service (NWS) has the primary responsibility for federal flood warning programs in the United States. The NWS provides specific flood forecast and warning services to over 3,100 communities and also works with many of the approximately 900 communities that have some form of local warning system (Federal Interagency Floodplain Management Task Force 1992). Over 21,000 communities have been identified by FEMA as being floodprone. The approximately 17,000 other communities receive warnings only through general county-wide flash flood warnings.

NWS flood forecasts are prepared and disseminated by 13 River Forecast Centers, each of which is assigned responsibility for one or more major river basins. Forecasts include the height and timing of the flood crest and the times when the river is expected to rise and recede to flood stage. Crest forecasts can be made a few hours in advance for communities on rivers draining small watersheds and two or more weeks in advance for downstream sites on large rivers. At many locations, particularly along larger streams, daily forecasts of river stage and/or discharge are routinely prepared.

The number of local flood warning systems has grown in recent years, and there is great potential for continued growth. Local warning systems are categorized as either manual or automated. A manual system usually consists of volunteer observers who relay data by telephone to a community flood coordinator. This person uses some kind of simple procedure, usually provided by the NWS, to convert the data to a river forecast stage. After consultation with NWS staff, the coordinator notifies the local response agencies. Staff gages located at pumping stations and gravity outlets provide valuable information for operations. Automatic systems consist of an automated data collection system, communications, data processing, and warning dissemination system. An automated local flood warning system may be as simple as an upstream river stage gage which sounds an alarm downstream at some predetermined stage, or complex enough to include satellite telemetry, sophisticated hydrologic modeling, and detailed response plans (Federal Interagency Floodplain Management Task Force 1992).

ALERT and IFLOWS are two commonly used types of operational automated local flood warning systems. The Automated Local Evaluation in Real Time (ALERT) system consists of automatic reporting rain and stream gages, radio telemetry, and computer analysis of the data. The NWS had the lead role in developing the ALERT systems, but they are available through private vendors as well as the NWS. ALERT systems have been adopted by many communities, particularly in the western United States. The Integrated Flood Observing and Warning System (IFLOWS) is a network of automated systems which uses federal, state, and local resources to provide detailed flood warnings to large regions with multiple political jurisdictions.

c. Flood Warning and Project Operations. Closure of levee openings and operation of gravity outlet gates and pumping plants in response to rising flood waters include the following activities:

- flood threat recognition,
- warning and mobilization,
- operating closure and interior facilities, and
- coordination between all components of the flood response effort.

Flood threat recognition is based on monitoring river flow, and interior runoff ponding levels and rainfall conditions and may also include forecasting of future conditions. As previously discussed, hydrologic monitoring systems range in complexity from a simple staff gage at the site of the local flood protection project to an elaborate network of automated precipitation and/or stream gages. In addition to providing information regarding current conditions, data from the monitoring system may also provide a basis for predicting flows several hours, days, or even weeks into the future. The value of a flood forecast depends on both accuracy and timeliness, which are inversely related. As discussed in Section 2-7(a), accuracy decreases with the length of time into the future for which the forecast is made.

Officials responsible for various aspects of project operations must be warned of the flood threat. Personnel must be mobilized and decisions must be made regarding when to close the various highway and railroad openings, close the various gravity outlets, and turn on pumps at the pumping stations. Communications and activation of personnel may occur during normal off-duty hours, under adverse weather conditions, with urgent time constraints. A flood preparedness plan is essential, with responsibilities clearly delineated and required actions outlined. Continuous plan management involving maintenance, drills, document updates are required to assure its viability.

After responsible personnel are mobilized and the pre-developed plan of action initiated, the time required to physically operate closure structures and interior facilities varies greatly between projects depending on many factors including:

- the number of facilities, their locations, and access,
- the types of facilities and their size, configuration, design, operating requirements, and readiness state,
- the number of personnel available to perform the work and their levels of skill and preparedness,
- warning disseminant message,
- weather conditions,
- interior water levels and stages in the vicinity of the facilities, and
- whether appropriate equipment and materials are readily available, well maintained, and functioning properly including the flood threat recognition system.

Examples of ranges of closure times for traffic openings are cited in Chapter 3. St. Louis has 31 closures with either single or double swing gates and seven other larger closures that require placement of panels. Crews of 2 to 7 people are dispatched to close the swing gates. After crews and equipment arrive on site, a single swing gate requires 15 to 20 minutes to close. Double swing gates require 30 to 90 minutes to close depending on the procedure used to brace the gates. For the seven panel closures, a crew of 12 to 20 people is assigned to construct the framework for the panels and then place the panels. A single row of panels is about six feet tall and requires 8 to 12 hours to erect. Higher rows of panels can be added as the river rises. The Columbus project is not yet operational, but the closure times shown in Table 2-3 for the 14 closure structures were estimated in conjunction with designing a flood forecast and warning system. A crew of 8 people is estimated to need about 38 hours to close all 14 openings, which include 5 gate, 5 stoplog, and 4 sandbag closures. The time is reduced by using multiple crews.

Chapter 3
Case Study: Flood Damage Reduction Projects in the Mississippi and Scioto River Basins

Several local flood damage reduction projects are presented here as case studies. They illustrate various configurations of closure facilities and interior flood damage reduction systems and highlight pertinent issues involved in their planning and management. Experiences and lessons learned in operating the projects during the record Flood of 1993 provide a major focus of the case study discussion of this chapter. Although the Corps terminology has evolved in some cases since the description were prepared, the conditions were retained as originally written.

3-1. Local Flood Damage Reduction Projects Reviewed

The Huntington, St. Louis, and Rock Island Districts prepared case study reports describing three projects located in Columbus, OH, St. Louis, MO, and Bettendorf, IA respectively. The Rock Island District also performed and documented a post-flood evaluation of the performance of ten projects during the 1993 flood. This chapter summarizes the information provided by the U.S. Army Corps of Engineers, Rock Island District (1994g and 1995d), St. Louis District (1994b), and Huntington District (1994f).

The levee projects reviewed in this chapter protect urban areas located in the Mississippi River Basin. Advanced engineering and design is presently being completed for a proposed construction project in Columbus, Ohio on the Scioto River, a tributary of the Ohio River. The others are operational projects located on the upper Mississippi River and tributaries in Iowa, Missouri, and Illinois. Severe flooding occurred in this region during the Flood of 1993.

The 12 local projects are listed in Table 3-1. Locations of the 11 projects that were operational during the 1993 flood are shown in Figure 3-1. With the exception of the project in Ottumwa, all are levee projects designed and constructed by the Corps of Engineers. All are operated and maintained by cities, except the South Quincy project which is operated and maintained by the South Quincy Drainage and Levee District.

3-2. The Upper Mississippi River Basin Flood of 1993

*a. **The Flood.*** The Flood of 1993 in the Midwest was a hydrological event without precedent in modern times (U.S. Army Corps of Engineers, North Central Division 1994e; Interagency Floodplain Management Review Committee 1994). It surpassed all previous floods in the United States in terms of precipitation, streamflow peaks and durations, area of flooding, and economic losses. From June through September 1993, record and near record precipitation fell on watersheds already saturated by previous seasonal rainfall and snowmelt, resulting in flooding along major rivers and their tributaries in the upper Mississippi River Basin. River levels exceeded flood stage at approximately 500 National Weather Service river forecast points, and record flooding occurred at 95 of the forecast points. The peak flow rate exceeded that of the 1% chance exceedance frequency flow at 45 U.S. Geological Survey gaging stations. Some locations remained above flood stage for five continuous months. The flooding duration was several weeks at many locations.

Table 3-1
Case Study Local Flood Protection Projects

City	River
Huntington District Report	
Columbus, Ohio	Scioto River
St. Louis District Report	
St. Louis, Missouri	Mississippi River
Rock Island District Report	
Bettendorf, Iowa	Mississippi River and Duck Creek
Rock Island District Post-Flood Evaluation Report	
Bettendorf, Iowa	Mississippi River and Duck Creek
East Moline, Illinois	Mississippi River
Milan, Illinois	Mississippi and Rock Rivers
Rock Island, Illinois	Mississippi River
Muscatine, Iowa	Mad Creek and Mississippi River
Burlington, Iowa	Flint Creek and O'Connell Slough of Mississippi River
Des Moines, Iowa	Des Moines and Raccoon Rivers
Ottumwa, Iowa	Des Moines River
South Quincy, Illinois	Mississippi River
Hannibal, Missouri	Mississippi River

A rare combination of meteorological patterns produced a convergence zone over the upper Midwest between the warm, moist air from the Gulf of Mexico, and the cooler, drier air from Canada. Excessive precipitation during April through July 1993 fell upon ground already saturated by a wet fall in 1992 and spring 1993 snowmelt, producing severe flooding in a nine-state area in the upper Mississippi River Basin. A basin map is provided as Figure 3-2. The flood was unique in its large area extent and duration and in the fact that the Mississippi and Missouri Rivers crested within the same week near their junction. As indicated in Figure 3-3, flood stage records were broken on the Mississippi River from the Quad Cities area to below St. Louis. Four of the projects shown in Table 3-1 and Figure 3-1 are located in the Quad Cities area, and 11 projects are located in or near the areas of record flooding shown in Figure 3-3. Record flows and stages also occurred on the lower Missouri River. The 1993 flood broke records set by the floods of 1965 and 1973. In some areas, stages were more than six feet higher than the highest levels previously recorded.

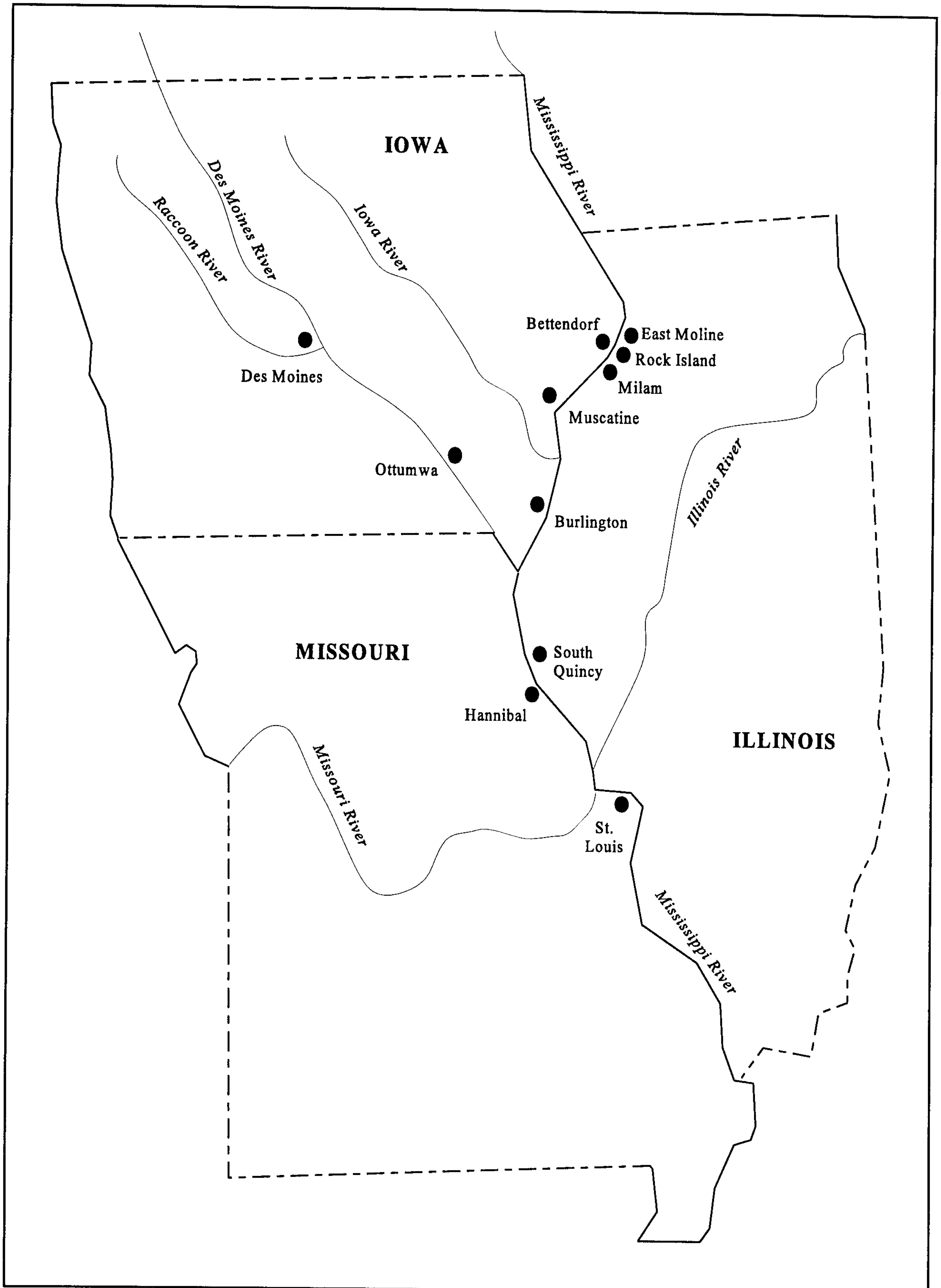


Figure 3-1. Location of Local Flood Protection Projects

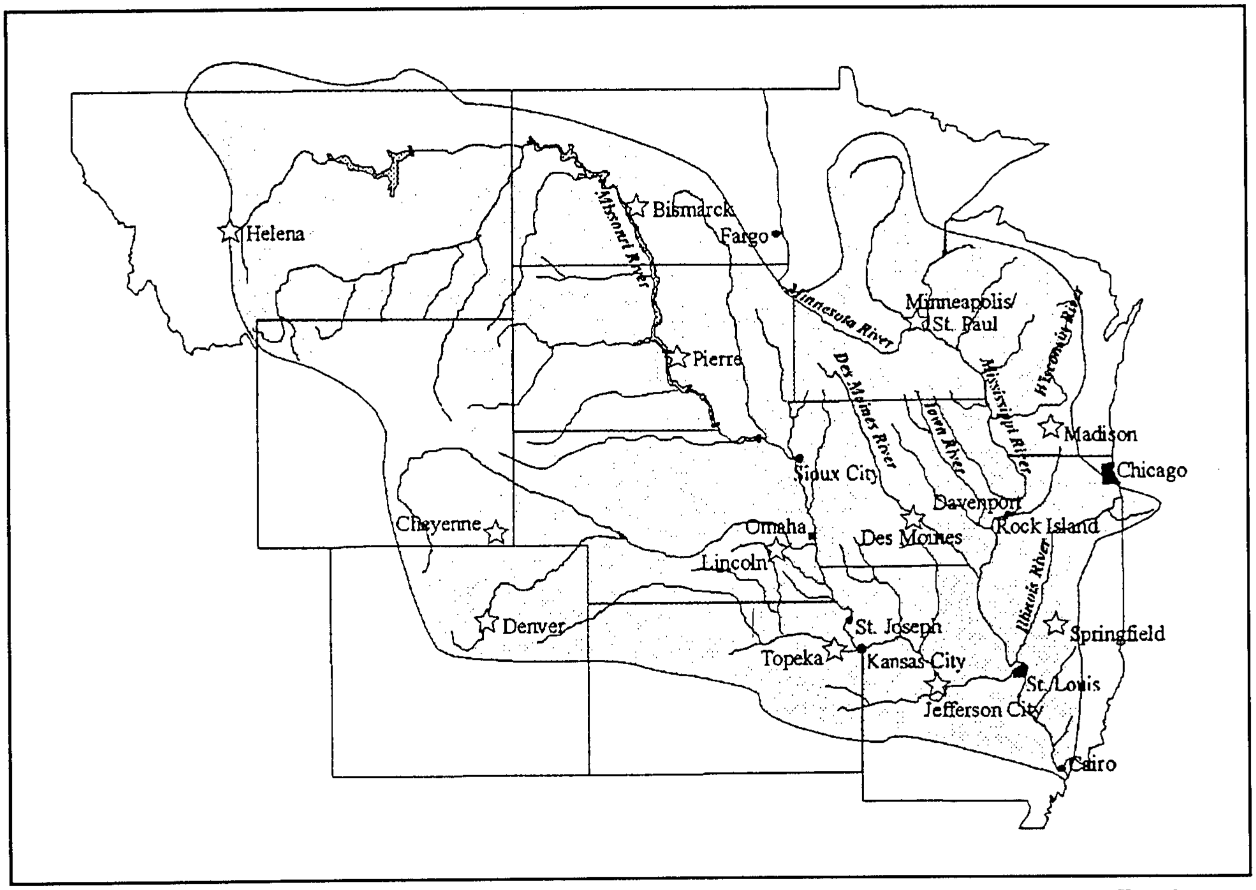


Figure 3-2. Upper Mississippi River Basin (Interagency Floodplain Management Review Committee (1994))

The flooding from the Mississippi and Missouri Rivers resulted in the death of 47 people and caused damages totaling between $15 and $20 billion (U.S. Army Corps of Engineers, North Central Division 1994e). Damage statistics include $6.5 billion in crop damage, 20 million acres of farmland damaged, 74,000 people evacuated, 72,000 homes damaged, 39 of 229 federal levees damaged, 164 of 268 non-federal levees damaged, 879 of 1,079 private levees damaged, and about 200 pumping plants and several water-treatment plants disabled. Navigation on the Mississippi River was stopped for 52 days. Major highways, bridges, and rail lines were closed for long periods (U.S. Army Corps of Engineers, North Central Division 1994e).

b. Flood Damage Reduction Measures. Most of the levee systems in this region were constructed by nonfederal entities to protect farmland. Towns later developed in areas protected by agricultural levees. The Corps of Engineers has been involved in strengthening and raising existing agricultural levees. The Corps has also constructed a number of local flood protection projects for protection of urban areas. As indicated in Table 3-2, the Corps of Engineers has constructed or improved over 2,200 miles of levees in the upper Mississippi River Basin, most of which are maintained by nonfederal sponsors (Interagency Floodplain Management Review Committee 1994). Nonfederal entities have constructed an estimated 5,800 miles of levees in the upper Mississippi River Basin. For most of the agricultural levees, overtopping flood levels are typically in the range of a .5 - 1% annual exceedance frequency. Many levees were overtopped during the 1993 flood. Almost no levees were breached except those that were overtopped.

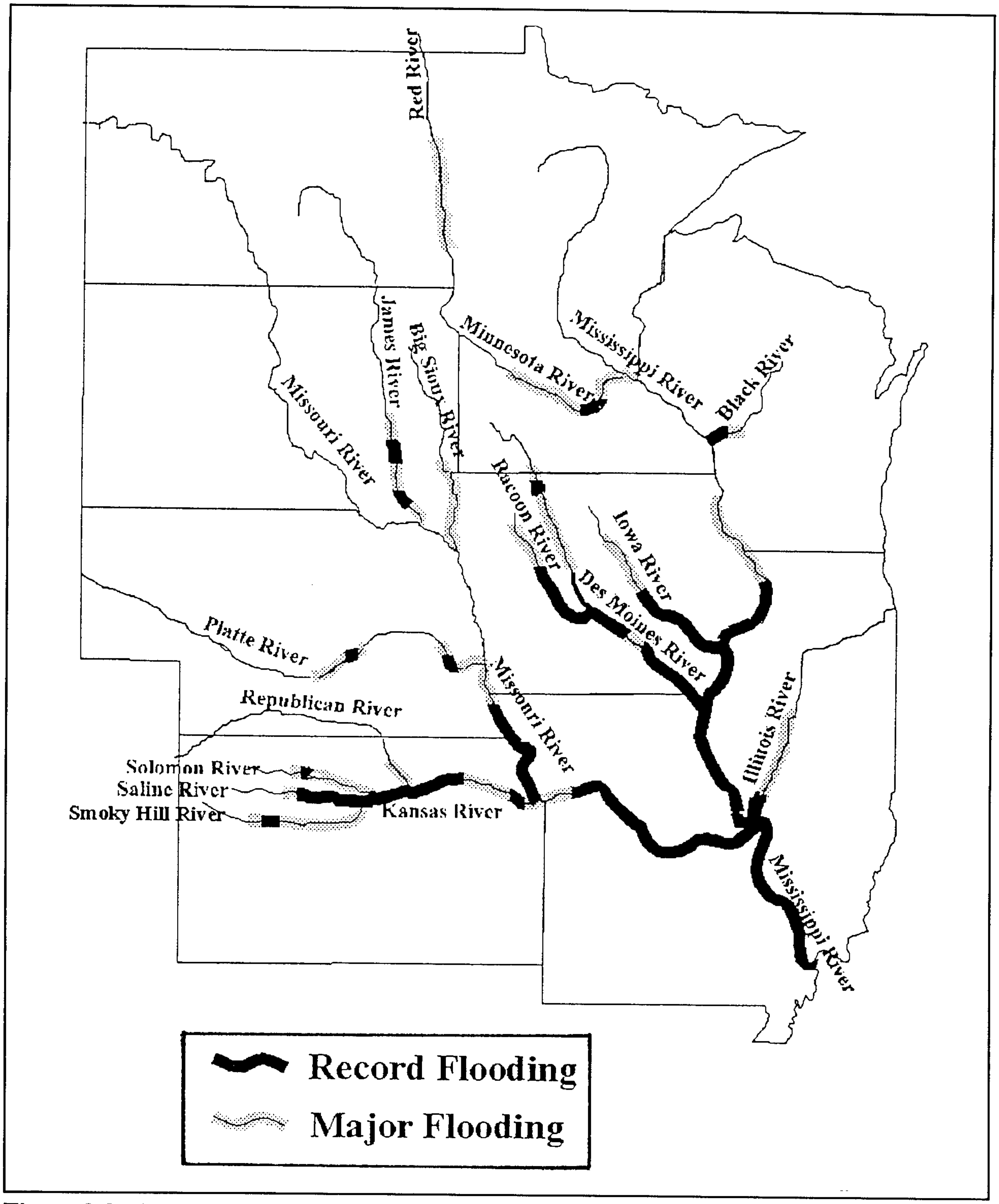


Figure 3-3. Areas Flooded in 1993 (Interagency Floodplain Management Review Committee 1994)

Table 3-2
Levees Constructed or Improved by the USACE in the Upper Mississippi River Basin (Interagency Floodplain Management Review Committee 1994)

River Reach	Corps District	Federal Maintenance (miles)	Local maintenance (miles)
Upper Mississippi	Saint Paul		17
Upper Mississippi	Rock Island	27	650
Missouri	Omaha/Kansas City	15	1100
Middle Mississippi	Saint Louis	___	440
Total Above Cairo, Illinois		42	2207

The Corps has constructed 76 reservoirs in the upper Mississippi River Basin. Most are on tributary streams to the Missouri River. The Corps also operates the flood control storage in 22 Bureau of Reclamation reservoirs in the Missouri River Basin. Several Corps reservoirs played significant roles in reducing flood damages during the 1993 flood, including the mainstem Missouri River reservoirs in Montana and North and South Dakota and three upper Mississippi River reservoirs.

*c. **Lessons Learned.*** Several studies, including two major floodplain management evaluations (Interagency Floodplain Review Committee 1994; U.S. Army Corps of Engineers 1995c) were conducted because of the 1993 flood. They resulted in policy recommendations for managing the Nation's floodplains.

The U.S. Army Corps of Engineers, North Central Division (1994e) post-flood report includes a summary of lessons learned from the Corps operational perspective. The lessons pertinent to the performance, function, and workability of closures and interior flood damage reduction systems are noted as follows:

- Corps staff responded with professional skill and personal dedication throughout the months of the flood. Few federal or nonfederal personnel had experience in dealing with extreme events, such as the magnitude of the 1993 flood. As the flood develops, operations personnel are necessarily in an accelerated learning mode. Covering every need in advance in preparation for such rare events is impractical. However, having resources available to respond to the emergency is invaluable.
- Agencies responsible for maintaining levees should be periodically informed of their responsibility to maintain a clear zone to allow for inspection and to prevent roots from forming channels under or through the levees. Where trees already exist, they need to be removed, including roots, to a specified depth and then the holes filled with impervious material.

- A video showing the filling, handling, and placing of sandbags and building of ring levees or berms and use of plastic sheeting should be prepared and made available to appropriate organizations and personnel.
- New techniques for flood fighting were used during the 1993 flood. Examples include portable floodwalls, called concertainers, and rubber bladders filled with water, known as water tubes. Flashboards were added to closure structures or floodwalls to prevent overtopping. In one instance, a levee was cut open to prevent a historic town downstream from flooding.
- The concept of using terms such as flood reduction, flood mitigation, or flood management, instead of flood control was reemphasized. Floods cannot be totally controlled. However, they can be greatly reduced and better managed with structural and nonstructural measures and actions.
- The lack of a single, integrated electronic data-storage system was evident. A better method is needed for transmitting current and accurate data in a timely manner regarding stages, flooded areas, precipitation, and other pertinent information. The use of a geographic information system (GIS) has been suggested.
- Needs exist for updating or otherwise revising stage-discharge and stage-damage relationships at Corps projects.
- Hydraulic and damage data must be collected immediately following the flood event.
- Coordination between the Corps and National Weather Service needs to be improved in regard to timely stage forecasts.
- An easy method needs to be developed for distributing basic hydrologic data to the public, state and local governments, and within the Corps.
- The news media needs to better coordinate their stories with agency technical personnel.
- Critical infrastructure, such as water treatment, sewage, and electrical power plants, need to be identified and their critical flooding stages established.

3-3. St. Louis District's Project at St. Louis, Missouri

When authorized in 1955, the City of St. Louis Project was the largest flood damage reduction project of its kind in the United States. St. Louis, Missouri is located on the Mississippi River a short distance downstream from its junction with the Missouri and Illinois Rivers. The Mississippi River at St. Louis has a drainage area of 697,000 square miles, draining all or part of 14 states and Canada. The project is comprised of two reaches, just north and south of downtown St. Louis, with the reaches separated by high ground. The project provides a high level of flood protection for over 3,000 acres of highly developed commercial and industrial area, along with over 200 miles of railroad track. The downstream and upstream reaches of the project protect about 2,530 and 630 acres, respectively. Both reaches protect long, narrow tracts of land, with the distance from the line of protection to high ground varying from a few hundred to 2,000 feet.

a. Project Features. The project includes seven miles of concrete floodwall; four miles of earthen levees; 38 closure structures for roads and railroads; 44 separate sewer systems of which 22 were diverted or pressurized to minimize pumping requirements; 28 pumping stations, ranging from 44 cfs to 4,000 cfs (average capacity of 400 cfs); 44 gravity outlets, ranging in size from 15 inches to a 23 feet horseshoe-shaped tunnel; about 11 miles of replacement sewers, ranging from 8 inches to 20 feet in diameter; and about 6,600 feet of pressurized sewers. Figure 3-4 is a map of the project.

The design discharge of 1,300,000 cfs was the estimated peak of the historic 1844 flood event. Current thinking is that the 1844 peak flow was actually about 900,000 cfs rather than the earlier estimate of 1,300,000 cfs. When the levee was authorized, the design flood magnitude was estimated to be a 0.5% annual exceedance frequency event. The discharge-frequency relationship has been re-evaluated over the past several decades, reflecting the addition of upstream flood control reservoirs, primarily in the Missouri River Basin. With the upstream reservoir control which came on line throughout the period from the mid-1950's through the 1980's, the level of protection is now estimated to be equivalent to a 0.2% to 0.1% chance exceedance frequency event.

Construction was initiated in 1959. The levees and floodwalls were essentially complete by the mid-1960's. The last of the interior flood control features were completed in 1975. The total cost of the project was $79,505,200, including about 2.3% non-Federal funds. The project is estimated to have saved over $900,000,000 in damages during the 1993 flood alone, which is many times greater than the entire first cost of the project. This flood was equivalent to a 0.7 - 0.5% chance exceedance frequency event. The project prevented damages estimated at $160,000,000 during the 1973 flood, the flood-of-record prior to the Flood of 1993. Significant damages were also prevented from floods occurring in 1969, 1979, 1982, 1983, 1986 and 1994.

b. Institutional Arrangements. The project was initiated with the formation of the St. Louis Flood Control Association in 1948, which worked to obtain project authorization. With a favorable feasibility report completed, the City of St. Louis passed a $7,500,000 bond issue in 1955 to pay for the local sponsor costs. The City entered into a Memorandum of Understanding (MOU) with the Corps of Engineers in 1956, officially becoming the local sponsor. The City agreed to the normal "hold and save" clauses and to operate and maintain the project. Upon project completion, the City of St. Louis Department of Streets has been assigned responsibility for the levee and floodwall, including the 38 closure structures for streets, highways, and railroads crossing the line of protection. The Metropolitan Sewer District (MSD) is the local agency responsible for sanitary and storm drainage in the City and County of St. Louis. Consequently, the City entered into a separate MOU with MSD to take over the operation and maintenance of the pumping stations. MSD has assumed responsibility for operating and maintaining the 28 pump stations and all sewer systems modified for the project. The City and the MSD work closely during flood situations. The Corps conducts periodic inspections of pumping plants and the line of protection, to note maintenance problems.

Both the City of St. Louis and the Metropolitan Sewer District mobilize their personnel during potential flood situations to operate different portions of the project. MSD operates the gravity drains, diversions and pumping plants, while the City handles the line of protection and makes the road closures. The Mississippi River at St. Louis can be forecast for several days in advance, thus allowing ample warning time to initiate closures or pumping. In addition, closures and pumping are initiated at varying levels. Some closures and pumping plants have only operated twice (during the 1973 and 1993 floods) in the more than 25 years that the project has provided flood protection.

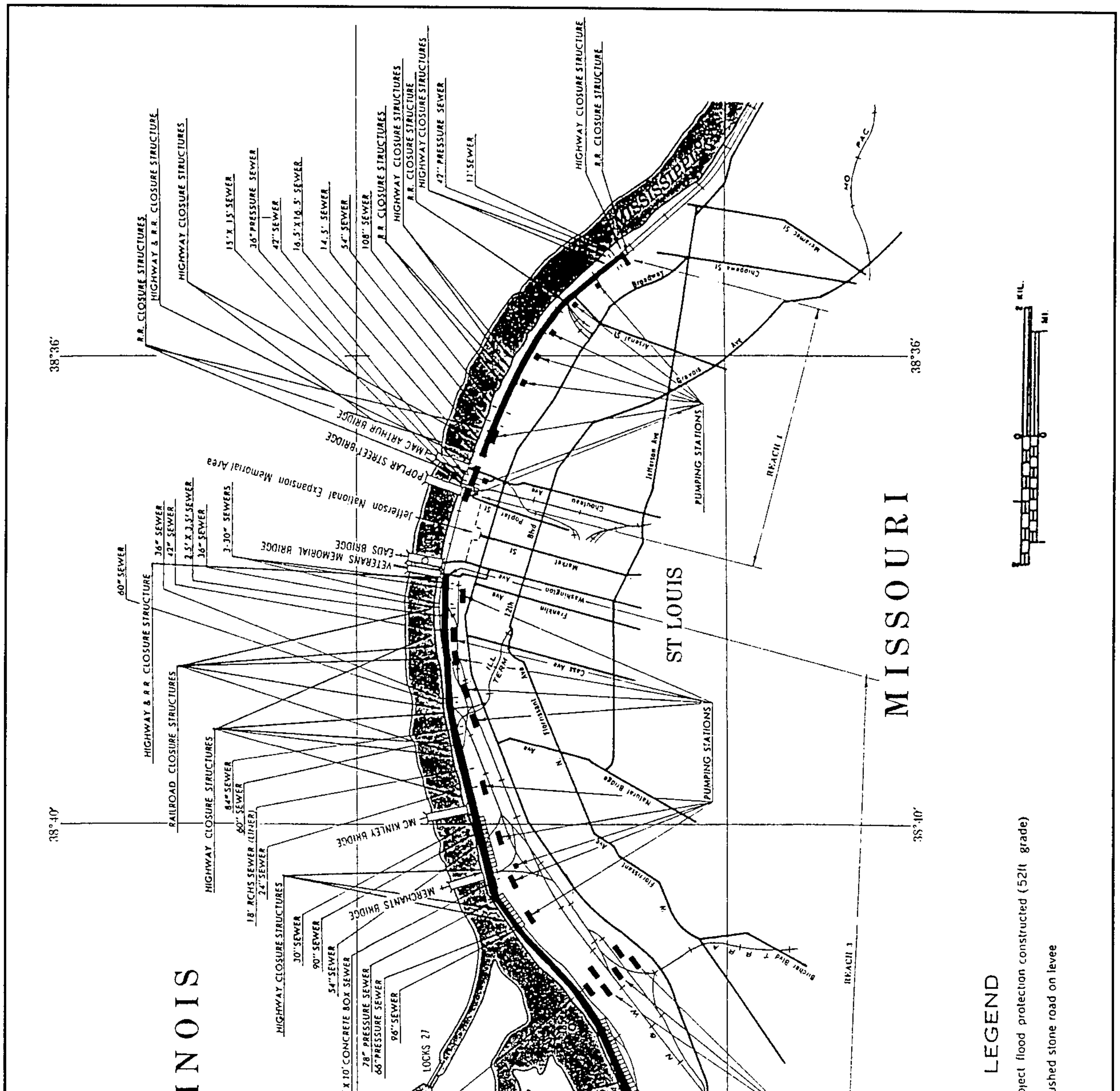


Figure 3-4. Local Flood Protection Project in St. Louis (U.S. Army Corps of Engineers 1994b)

The City uses a private weather forecast service to supply weather predictions, with this service having access to National Weather Service river forecasts. The Metropolitan Sewer District maintains contact with the City and with the National Weather Service for river stage forecasts.

c. Closures. Gate installation for closure of street, highway, and railroad openings through the line of protection is initiated when stages are forecast to exceed the elevations of the gate sill. Thirty-one gate closures are either single or double swing gates. A single swing gate requires 15 to 20 minutes to close, while a double swing gate requires 30 to 90 minutes, depending on the procedure used to brace the gates. Small crews of from 2 to 7 personnel are dispatched to make the swing gate closures. Single swing gate closures are used for openings up to about 22 feet in height and 20 feet in width. Double swing gates are employed for opening widths of 20 to 40 feet and heights up to 16 feet. Seven panel closures are required for the wide openings. A crew of 12 to 20 personnel is necessary to construct the framework for the panels. A single row of panels is about six feet high and requires 8 to 12 hours to erect. Higher rows of panels are installed as forecasts show higher river levels. These reinforced aluminum panel closures are used for openings of 40 to 70 feet and for heights up to 20 feet.

d. Interior System. Areas of about 16,000 and 7,418 acres, respectively, drain through the interior flood damage reduction systems to the Mississippi River in the downstream and upstream reaches. The interior areas are essentially 100% storm sewered, except for some cemeteries and park areas. No significant surface streams exist and all interior areas are fully urban. With all the runoff channeled through storm sewers, rate of rise is rapid and durations are short at the line of protection. Since essentially no surface storage is available, the pumping plants receiving non-diverted flow are pumping the peak inflow to the site. With blocked drainage and a storm exceeding the interior design (estimated 3.3% chance exceedance frequency event), some street and basement flooding may be experienced for a brief period. However, this situation has not occurred to date.

Gravity drain closures are made and pump stations are staffed as the river rises. Pumping plants are manned at stages ranging from 27 to 39 feet at the St. Louis gage. During floods, staffing around the clock is necessary. During moderate floods (4-5 feet over flood stage of 30 feet, about a 20% chance exceedance frequency), over 40 Metropolitan Sewer District personnel are assigned to the various pumping plants that may require operation at this river stage.

e. Operation and Maintenance Experience. The project has provided flood protection since the mid-1960's. Construction was completed on the last interior feature in 1975. Eight significant floods (20% chance or rarer) have occurred, including the Flood of 1993. No significant damages to the protected area resulted from any of these events, either from the river or from interior runoff.

As previously discussed, the project is operated and maintained by the City of St. Louis and Metropolitan Sewer District. The annual budget for OM&R for the project is not broken out by either MSD or the City. However, MSD's rough estimate of its annual O&M expenditures during a non-flood year is in excess of $100,000. During a moderate to severe flood year, expenses are many times this figure. The City's estimate of its past annual O&M expense is about $25,000 during a non-flood year, which is insufficient to properly maintain all necessary items. During the 1993 flood, emergency contracts let by the City to stabilize the Riverview flood wall and Salisbury Pump Station areas, discussed later, required more than $400,000. City manpower costs to monitor the project were an additional several hundred thousand dollars. Final repair costs to restore the project to its original operating conditions are an estimated $1,462,000, which is a Federal rather than a local sponsor cost.

As the project has gotten older, additional maintenance is required. However, due to funding problems of the local sponsor, this maintenance has often been deferred. The floodwall and levee have functioned adequately, considering that minimal maintenance has been performed. However, deferred maintenance due to funding problems is taking a toll on potential project effectiveness and resulted in major problems during the 1993 flood.

The City conducts annual drills of closing all swing gates to ensure successful operation during floods. The panel closures are not made, due to the time and expense. Routine mowing of levees and some tree and brush removal is performed, along with periodic painting of gates and other items. Theft and vandalism are problems. Some of the aluminum panel closures have been stolen and sold for the metal value, forcing the acquisitions of replacements. Panels and other materials necessary for closures are now stored at a centralized location for security.

Major problems with the pumping stations have not been experienced. Although the pumping stations have been in operation for 15 to 30 years, no major overhaul or replacement has been necessary. This circumstance may be attributed to the limited operation of the main storm water pumps. Some of the small base flow pumps, for sanitary discharge, have been replaced by submersible units. Maintenance of the interior flood damage reduction system has generally been good, with few problems noted by the St. Louis District during inspections of the facilities. The rating for the interior facilities in effect prior to the Great Flood of 1993 was "Fully Acceptable".

Prior to the flood, the rating of the floodwall portion of the project by the St. Louis District was "Minimally Acceptable", due to unresolved maintenance problems. Unresolved maintenance problems noted during past Corps of Engineers inspections of the project include:

- significant tree growth in levee and near floodwall, causing potential flow paths through the levee;
- significant tree growth in close proximity to relief wells, likely causing crushing of the well casing;
- lack of periodic flushing of relief wells (levees) and sub-drainage systems (floodwall), possibly rendering these systems ineffective during severe floods;
- broken or paved over relief wells/piezometers;
- rehabilitation of under-drainage system;
- ungated/abandoned conduits through the line of flood protection;
- severe rusting of the bottom of some swing gates, as well as the hinge pins for others;
- inability to make closure at certain railroad crossings, where the track has been re-leveled over the years and the rails are now higher than the bottom of the closure;
- and spalling of concrete on some structures.

These problems caused difficulties during the 1993 flood ranging from minor to major in nature. The problems noted during the 1993 flood and the concerns of insurance companies that insure properties protected by the project may be sufficient to increase the level of maintenance performed in the future. The City currently plans to increase maintenance expenditures for the project, especially to test and rehab the relief wells throughout the system.

The monitoring and limiting of encroachments riverward of the levee which would adversely affect the design grade, have been a continuing problem. With a general desire by industries to have ready access to the river for loading and unloading operations and from the City to better use city-owned land, some encroachments have occurred over the years. Limited fills have been permitted which, along with other encroachments over which the Corps does not have permit control, have undoubtedly resulted in a slight increase (0.1-0.3 ft) in the water level of design flow at certain locations.

A significant modification to the function of the project was required to comply with Environmental Protection Agency requirements preventing the release of untreated combined storm and sanitary sewer flows to the river. All the older sewer systems in the protected areas feature combined sanitary (base flow) and storm water flow. All base flows are intercepted and pumped to treatment plants regardless of river stage. During larger storm events, the combined flows may be released to the river, because of the large volume of interior runoff available for dilution with the base flow.

f. Lessons Learned. With hindsight and with newer technology, there are several things that would be done differently today. The U.S. Army Corps of Engineers, St. Louis District (1994b) queried District staff and representatives of the City of St. Louis and the Metropolitan Sewer District for their thoughts on this subject. Some of their responses are discussed in the following paragraphs:

1. *Minimum Facility.* Today's criteria require a minimum facility to be determined and then any additional interior flood damage reduction measures to be incrementally economically justified. The minimum facility for the original project would likely have featured gravity drains and pressure sewers, with minimal pumping facilities. It is likely that after further analysis, the resulting pumping plants would have been smaller if incremental economic justification had been required during feasibility and design.

2. *Hydrologic Criteria.* The design approach of assuming one or two coincidental design storms is wholly inadequate for today's interior flood control criteria. Coincident frequency analysis and/or continuous simulation analysis would have been performed to identify the complete interior stage-frequency relationship for each interior area. Interior hydraulic modeling using the Storm Water Management Model (SWMM) or a similar sewer routing program may have been necessary for proper evaluation. The infrequent operation of many of the pumping plants suggest that current analysis standards would likely have resulted in much decreased pumping capacities.

3. *Freeboard.* The original design incorporated a uniform two feet of freeboard throughout the protected reach. A better design would incorporate the modern standards of a variable freeboard to insure overtopping at the downstream end of the protected reach or least hazardous location.

4. *Type of Pump.* The use of submersible pumps, located within the gravity drains at the line of protection, would be emphasized to eliminate many of the pumping station buildings. This type of pump would result in a significant decrease in both first cost and annual O&M cost.

5. *Trash Racks.* Trash racks on pumping stations are designed to prevent the passage of material larger than three inches into the pump intake line. Only the two largest pump stations have mechanical trash raking equipment to keep the screens free of debris buildup. The other 26 stations require manual removal of debris. This results in continuous maintenance problems in keeping the screens reasonably clear for unobstructed flow into the station. Mechanical trash raking equipment and/or larger screen openings would minimize this problem and decrease the existing O&M cost to operate the stations.

6. *Increased Automation.* Although much of the interior system has been automated since completion of construction, designing as much of the system as practical for automated openings of drains, pressure sewers, and pumping would have resulted in long-term O&M savings.

7. *Improved Closure Design.* Simplification of closure mechanisms to minimize the amount of time required to close would improve the operation. In addition, railroad closures could be designed with a section, perhaps 50 feet or so, of removable rail, to facilitate railroad closures being installed quickly and relatively easily.

8. *Better Maintenance Controls.* No real mechanism exists to enforce maintenance requirements. The inability to provide all necessary maintenance nearly resulted in a disaster during the 1993 flood.

g. Floods other than 1993. Even before completion, the project prevented extensive damage during floods in 1969 and 1973. Prior to the Great Flood of 1993, the 1973 event was the flood of record, reaching a stage of 43.3 feet at the St. Louis gage (13.3 feet above flood stage) and occurring prior to the start of construction of the last pumping plant. For this pumping station, a 300 feet long gap existed in a flood wall which was successfully sandbagged to prevent flooding. Additional damage was prevented during floods in 1979, 1982, 1983, 1986 and 1994; all of which crested at stage 37-39 feet on the St. Louis gage. Mississippi River floods are marked by generally slow rises and falls and minimum durations of one week above flood stage (30 feet, St. Louis gage). Several weeks duration above flood stage is not uncommon and both the 1973 and 1993 floods remained above flood stage for 11 weeks or more.

h. The Great Flood of 1993. This flood was extraordinary in every facet of hydrology and hydraulics, and severely tested the project. The exceedance frequency for the 1993 flood is estimated to be between 0.7% and 0.5% years. The river crested at 49.6 feet at the St. Louis gage, leaving 4 to 5 feet to the top of the floodwall.

The Flood of 1993 was remarkable for its peak discharge, volume, duration, time of year it occurred, and peak stage. It almost certainly is the greatest flood event occurring at St. Louis in the more than 200 year history of the city. The peak discharge of 1,070,000 cfs exceeded the previous record discharge by 20%. An earlier estimate of the peak flow of 1,300,000 cfs for the 1844 flood, in which discharge was not measured, is higher than the 1993 flood. However, this historical value is believed to be over-estimated, with the actual peak flow of the 1844 flood being less than 900,000 cfs. Peak stage exceeded the record stage by more than six feet and generally came within 4 to 5 feet of the top of the floodwall/levee. Approximately 112 million acre-feet of water passed St. Louis during the main portion of the flood from June 26 to September 13. This volume represents a uniform depth of three inches over the entire 697,000 square mile watershed upstream of St. Louis, or a depth of 17 inches over the two state area of Illinois and Missouri. Mississippi River floods at St. Louis usually occur in the spring or early summer, but the 1993 flood was a July-August-September event, the first time this had happened. Duration was perhaps the most remarkable statistic of the flood. The river at St. Louis exceeded flood stage for 104 consecutive days and for 148 days during the calendar year. The previous record for both durations was 77 days in 1973. During the maximum portion of the flood, the river was 10 feet or more above flood stage for 36 days, exceeded the "2% chance flood" level for 23 days and the "1% chance flood" level for eight days. Before 1993, there were only 12 days total in the entire period of record, dating back to 1861, that exceeded flood stage by 10 feet or more. The Great Flood of 1993 was obviously a worthy test of the St. Louis Project.

i. Performance During the 1993 Flood. The project performed extremely well, with exceptions noted in subsequent paragraphs. The project was successful in preventing flooding to a very valuable area. Unforseen problems arise during emergency situations, requiring a quick and correct response. This was the case during the Flood of 1993, and these responses allowed the project to function and protect the area. Maintenance of the project must be improved in the future, to prevent any reoccurrence of problems similar to those discussed later.

Gate closures were made promptly, and all pumping stations were staffed and operated as necessary. Minor leakage occurred at many closures, but was not a significant problem. Seepage at closures was reduced or stopped by milled asphalt sealing riverward of each closure. Of more concern was inflow via several abandoned and ungated conduits through the line of protection. Sealing of these entry points often required divers to manually place sandbags or other materials in the openings on the riverside of the levee or floodwall.

All pumping stations performed adequately, with no significant interior flooding resulting. The only problems with pumping plants occurred due to loss of electrical power from lightning strikes. Power was returned within one or two hours. Some minor basement flooding occurred as a result of these pump outages.

Seepage and sand boils were a constant problem due to the abnormal duration of high heads. These were ringed as necessary to decrease the head differential across the line of protection.

The two serious problems that occurred are discussed in the following paragraphs. A near failure of a floodwall was caused by an undermining of a reach of the floodwall by seepage. Emergency actions prevented failure of the floodwall and resultant tremendous flood losses. The other major problem involved subsidence at a pump station caused by leaking pipes.

At 2300 on July 22 with the St. Louis Gage at 46.7 feet, the Emergency Operations Center at the St. Louis District Office received a phone call that a major sand boil had occurred at the northern end of the project, near Riverview Drive. By 0045 on July 23, the boil had erupted into a geyser, estimated as 5 to 6 feet high and having a flow rate of several cubic feet per second. A vortex was apparent on the riverside of the floodwall, indicating the severity of the situation. Fortunately, representatives from the City and the Corps quickly brought in 100 tons of rock that was placed over the leaking area by 0230 on July 23, slowing the leak to about one-third of its previous flow. The vortex was reduced in size and was no longer active by 0600 on July 23. Soundings made along the river side of the floodwall on the morning of July 23 showed a 70 feet long trench had developed with depths ranging up to 17 feet deep. More than 2,000 tons of additional rock and other material were placed on both sides of the wall throughout the day, which further reduced inflow. During the afternoon of July 23, a spall appeared at the top of the wall at a junction, and a section of the wall began to tilt toward the river. At nearly the same time, the next 20 feet long section of floodwall also began to move riverward at an even greater rate. A vertical crack appeared at the base of the wall, which developed into a full split, leaking water at several locations. A sinkhole appeared behind the wall and inflow increased greatly. Rock fines were immediately placed into the sink hole until it was filled and seepage was reduced to less than one cfs. The wall movement continued at a very reduced rate, with final riverward movement of two 20 feet sections of some 3.25 and 2 inches, respectively. By 1600 on July 23, the under-seepage had been reduced to approximately 0.1-0.2 cfs and wall movement had virtually ceased. Soundings taken at 1700 showed the trench to be filled with rock. The following few days saw around the clock construction of a rock ring levee landward of the troubled section of flood wall, then the drilling of 12 grout holes riverward of the wall through the footing. Approximately 111 cubic yards of grout were placed into cavities below the floodwall footing by July 30 (two days before the crest), stabilizing a highly dangerous situation. Although the exact cause of this problem may never be positively proved, it appears to have resulted from a rusted-shut flap gate on the under-drainage system immediately landward of the floodwall. Pressure built behind the gate with the rising river until the pipe burst, allowing a flow path from the river to the protected interior. Had the emergency operations not been successful, the floodwall would have been undermined and eventually overturned, allowing the protected area in this reach to be flooded with the resulting damages in the hundreds of millions of dollars.

The second of the two major problems involved subsidence at the Salisbury Pump Station. The pump station is located near the lower end of the upstream reach. Minor subsidence was first noted prior to the onset of the flood, and plans had been made to rectify the problems when the river reached a low water condition, allowing the work to be performed. Unfortunately, the river never dropped to the required low stages until well after the flood. The subsidence was caused by separation of pipes just prior to entering the pump station. Material entering the pipes was pumped with the seepage and storm water runoff to the river, resulting in settlement of the ground surface near the pump station. By July 23, rock was being placed over cave-in areas near the pump station. This settlement extended around the pump station, across a parking lot and under a railroad track, making it inoperative. Up to two feet of settlement occurred around the pump station and extended into the levee landside toe, causing some levee slope failure to occur. The levee settled six inches at the landside toe, two inches just down the slope from the landside crown, and 0.5 inches on the riverside edge of the crown. Cracks formed in the levee crown and the landside slope. Rock was continually placed over areas of subsidence throughout the period, with as much as 2,000 tons placed during one 24-hour period. During the last week of July, a 42-inch pipe bringing much of the water and material to the station was grouted in an attempt to lower inflow to the station. If the flow could be fully shut off, the wet well at the station could be pumped down, allowing emergency lining of the other damaged pipes to take place. Unfortunately, the 42-inch pipe could never be fully sealed off by grout and the resulting leakage prevented the wet well from being pumped dry. It was decided to allow the water level in the wet well to rise, slowing the inflow rate, and accepting minor interior flooding should significant interior runoff occur. Fortunately, local rainfall was minor during the following period. Rock was placed on both sides of the levee for stability and over areas of subsidence. Although this situation was serious, it was not as dangerous as the Riverview floodwall problem.

Repairs at the two sites of major problems, associated with the near-failure of the floodwall and the subsidence at the pumping plant, were performed in 1994. The repairs to the floodwall near Riverside Drive involved removal and replacement of 80 feet of floodwall, including the under-drain system. Repairs at the Salisbury Pump Station, consisted of grouting the two existing leaking pipes for about 250 feet and replacing them with a single 72-inch pipe to the pump station. Fill material was placed at all areas of settlement, and the entire area restored to pre-flood conditions.

3-4. Rock Island District's Project at Bettendorf, Iowa

The City of Bettendorf, Iowa with a population of 30,000 people, is one of many communities forming an urban area on both the Iowa and Illinois sides of the Mississippi River, with a total population of about 380,000. The Mississippi River at Bettendorf has a drainage area of about 88,400 square miles. The city borders the river for 2.3 miles. Duck Creek forms the upstream boundary of the project. At its confluence with the Mississippi River, Duck Creek has a drainage area of 64.5 square miles. The local flood protection project was authorized in 1968 and constructed during the period 1982-1987. As the local sponsor of the federal project, the City of Bettendorf is responsible for operations and maintenance.

The project consists of earthen levees, floodwalls, closure structures, interior flood mitigation facilities, and pumping stations. The levee is sized to provide protection against a design flood with a 0.5% annual exceedance frequency, with three feet of freeboard. The project protects a 325 acre area, that is primarily industrial and commercial with some residential development.

a. Levee and Closures. The general plan for the project is shown in Figure 3-5. The line of protection is three miles long and consists of: 0.1 mile of downstream tie-off; 2.3 miles along the Mississippi River; and 0.6 mile of upstream tie-off along Duck Creek. The 0.1 mile long downstream tie-off section consists of (1) a length of 4-foot high concrete wall, with two openings with swing gates for railroads, and (2) a length of opening including a street crossing that is closed with sand bags during extreme flood events.

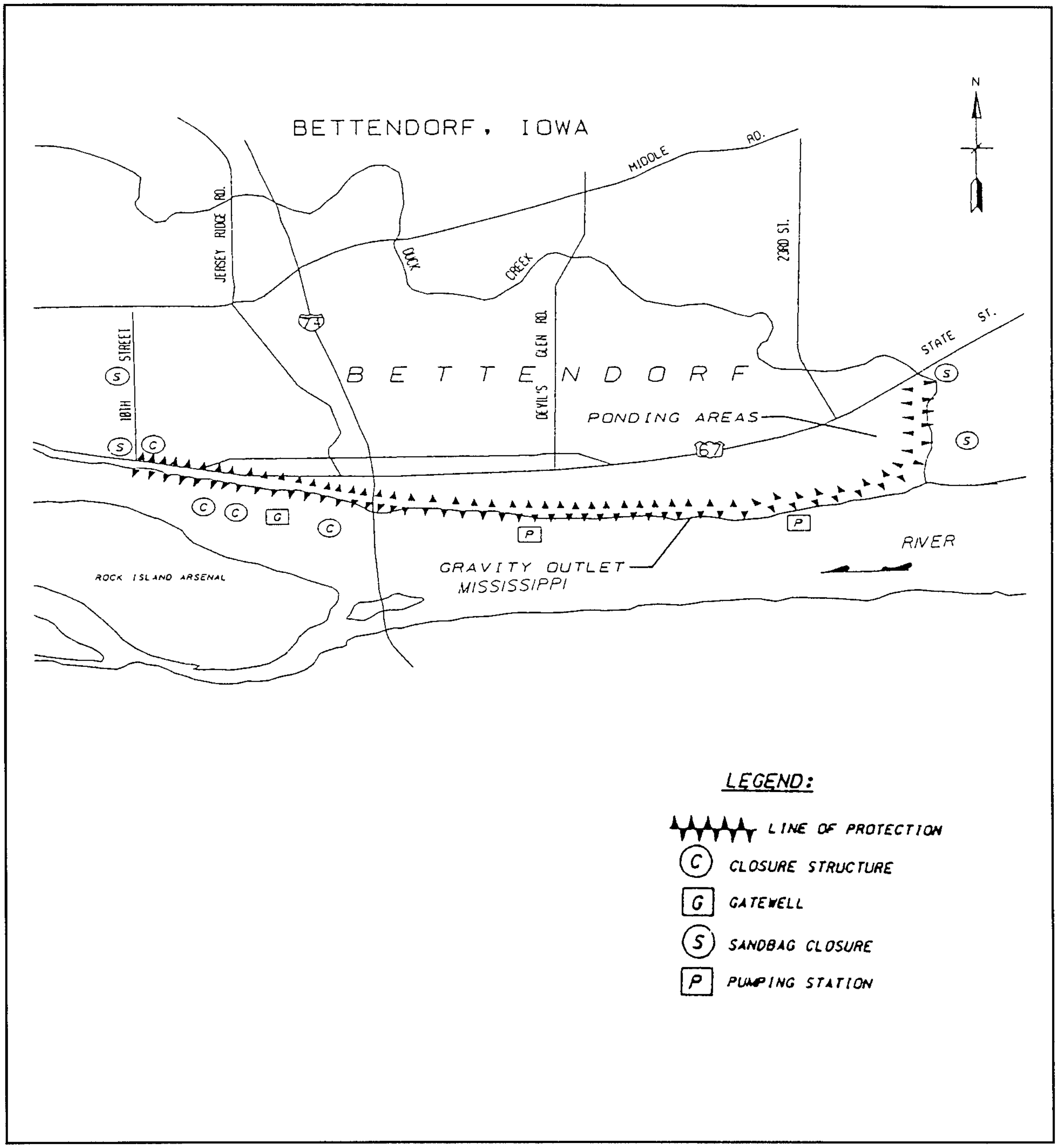


Figure 3-5. Local Flood Protection Project in Bettendorf (U.S. Army Corps of Engineers, Rock Island District 1995d)

The 2.3 mile main levee along the Mississippi River is earthen except for two sections: (1) a 325-feet long dock made of sheet pile cells and (2) a 220-feet long opening. The wide opening provides a view of the river during low stages as well as a street crossing. People picnic and launch boats in this area. Folding walls are provided on both sides of the street. Concrete walls tie the folding walls into the levee. During floods, the folding walls are erected, and the street is closed with a bulkhead gate.

The 0.6 mile upstream tie-off, following the bank of Duck Creek, is a combination of levee and concrete floodwall. Duck Creek was widened, and two railroad bridges were rebuilt, in conjunction with the project. This lowered the tie-off levee height. A street opening is provided that is above the 0.5% exceedance frequency design flood elevation but below the levee crest. This opening is closed with sand bags.

b. Interior System. The City of Bettendorf is located just upstream of Lock and Dam 15. A 3-mile long interceptor sewer, called the Government sewer, runs parallel to the Mississippi River to a discharge location downstream of Dam 15. The Government sewer was constructed in the 1930's to convey combined sanitary and storm runoff from the city. The later local flood protection project included addition of pumping capabilities to handle the blocked flow of the sewer during floods. Two pump stations discharge interior flood water across the main levee. During low river stages, the Government sewer conveys, by gravity flow, runoff from feeder storm sewers in the protected area. During high river stages, the sewer does not drain by gravity. If this condition occurs, the sewer is closed and a pumping station is placed into operation. The pumping station has a design capacity of 150,000 gallons per minute and no gravity outlet. The pump house is located over the Government sewer and contains 5 identical 36-inch pumps. The pumps discharge directly to the river through 34-inch diameter lines fitted with flap gates. Once started, the pumps are operated by automatic gate controls. When the pumps are not being used, inlet valves separate the pump chamber from the Government sewer, and a sump pump keeps the main chamber dry.

A second pump station serves another part of the interior flood damage reduction system. This station has three identical 24-inch pumps providing a capacity of 40,000 gallons per minute. From each pump, 24-inch diameter discharge lines run over the levee and discharge into a gatewell on one of the gravity outlets. The pump station withdraws water from an adjacent pond which is fed by to other remote ponds. Four gravity outlets adjacent to the pump station drain interior runoff during low Mississippi River stages. The 6 feet wide by 4 feet high conduits have gatewells with motor operated slide gates. Each pump forebay also has a motorized slide gate to isolate it from the pond area. A sump pump keeps the chamber dry while the main pumps are isolated.

Seven drainage structures with gatewells cross the line of protection. Five are along the main levee, and two are along the Duck Creek tie-off.

c. Flood Warning and Monitoring System. Flood forecasts are issued by the National Weather Service River Forecasting Center in St. Paul, Minnesota, Corps of Engineers, and local television stations. The upper Mississippi River has a slow rate of rise. Operating personnel are expected to receive a 5 to 7 day notice of the need to close gatewells, install closures, or to evacuate. Evacuation can occur along many roads that lead to high ground. However, Duck Creek has a much quicker rate of rise.

A water level monitoring system was installed in 1988. The automated system measures water levels at selected locations in the river and interior system and activates alarms whenever preset triggering levels are reached. The alarms are displayed at a dispatcher's desk in the police station, which is the only city office open 24 hours a day. An automatic dialer also telephones the numbers of key personnel. Problems have occurred with false alarms due to malfunctioning of the equipment. Better sensors have been installed to help eliminate false alarms.

d. Performance During the 1993 Flood. The maximum stage during the 1993 flood was about four feet below the design flood and seven feet below the top of levee. The project is considered to have performed very successfully. The flood was a learning process that resulted in several subsequent improvements. Several of the relatively minor problems that occurred are noted below.

Previously during construction, the contractor had tested the folding wall closure without problems. However, the 1993 flood was the first time that city employees unfolded the wall. As the bottom tier of the folding wall was being placed, removable clevis pins started to be pulled from the wall. The connections were repaired after the flood, and the operating instructions revised to specify two pieces of equipment, instead of one, for moving the folding wall. The flood water did not rise high enough to test the seals on the folding wall.

At another street closure, the gate did not seal properly and leaked even with sandbags added. Portable pumps were used to discharge the leakage back into the river. Either the force on the gate due to the 2.8 feet of water was not enough to compress the seals or the gate was incorrectly installed. Training on closure operations is needed.

Swing gates at two railroad closures performed as designed. The sand bag closures on both tieoff levees were not necessary during the 1993 flood and have never been built even as a training exercise.

Interior ponding areas are a concern due to siltation, vegetation growth, flat slopes, and access to ponding areas. A pre-ponding siltation pond would provide a potential solution to these problems.

During a previous flood in 1990, a power loss prompted the City to install an emergency generator for one of the pump stations. A trash rack was also added to the other pumping station.

Closure of gate wells was sometimes difficult due to rocks, silt, and woody debris in the path of the gate. Procedures for closing gatewells were uncertain due to the newness of the project.

The city purchased six 6-inch pumps and rented several additional pumps to help with the flood waters. The City staff felt fortunate that more severe interior rainfall events did not occur simultaneously with the high river stages.

The flow in sanitary sewers was also a major concern for the City. The sanity flow from Bettendorf is conveyed to the City of Davenport's treatment facility. Overflow in the sewer system would have occurred had the treatment facility been shut down. Also, much pumping was required to convey sanitary sewer flow.

3-5. Rock Island District's Post-Flood Evaluation of Ten Projects

The Bettendorf project and nine other local flood protection projects were included in a post-flood performance evaluation conducted by the U.S. Army Corps of Engineers, Rock Island District (1995d). These projects are listed in Table 3-1. Their locations are shown in Figure 3-1. East Moline, Milan, and Rock Island, Illinois, and Bettendorf, Iowa are located in the Quad Cities area along the Mississippi River. The cities of Muscatine, Burlington, South Quincy, and Hannibal are located on the Mississippi River between the Quad Cities area and St. Louis. Des Moines and Ottumwa are on the Des Moines River. With the exception of the Ottumwa project, these federal levee/floodwall projects were all designed and constructed by the Rock Island District. At Ottumwa, the Rock Island District made relatively minor modifications to interior flood damage reduction facilities for an existing system that had been constructed by local interests. All of the projects are maintained and operated by local sponsors. The U.S. Army Corps of Engineers, Rock Island District (1995d) reviewed the operation of warning systems, gate and sandbag closures, pump stations, ponding, and other interior flood damage reduction facilities. The reviews included interviews with local officials, engineers, and operations and maintenance personnel responsible for each project as well as review of planning and design documents.

The Bettendorf project is discussed in Section 3-4. The other projects are briefly described below. The information collected by the Rock Island District regarding performance during the 1993 flood is then summarized.

a. East Moline. The project was authorized in 1968, and construction was initiated in 1979. The levee and floodwalls were sized for a 0.5% chance exceedance frequency flood. Streets and railroad tracks are ramped over the levee except that. An opening with a gated closure structure is provided for one street. The interior system includes ponding areas for temporary storage of runoff, gravity outlets for discharge into the river at low stages, and a pumping plant for use when gravity flow is blocked. The pumping plant handles runoff from a creek with a large drainage area. The pumps have relatively little impact on the level of interior ponding, but rather are for evacuation of ponded water within a reasonable time after a flood event. Gatewells are provided on drains and sewers that pass through the levees and floodwalls. To reduce the number of drains passing through the levee structure, some drains are connected to interceptor sewers that discharge into gatewells.

b. Milan. The project was authorized in 1968, and construction began in 1981. The project is located on the Rock River between river miles 0.8 and 5.6 above the confluence with the Mississippi River. The project consists of 10.6 miles of levee and 1,120 feet of floodwalls with appurtenant closures, ramps, and interior flood damage reduction facilities including two pumping plants. Levees are sized to provide protection against 0.5% chance exceedance frequency floods on the Mississippi and Rock Rivers and several local creeks. A flood warning system was installed by the Corps of Engineers in 1988 in the Mill Creek watershed to alert the City of Milan of flash floods. The Milan levee project, which protects primarily urban areas, is connected to the Big Island Conservancy District levee, which protects mostly rural lands. The two projects were constructed at the same time, and their operation and maintenance are shared.

c. Rock Island. The project was authorized in 1962. The completed project was transferred to the City of Rock Island in 1980. The levees and floodwalls were sized for a 0.5% chance exceedance frequency event on the Mississippi River. Gatewells are provided on drains and sewers that pass through the levees and walls. Closure structures are provided for street and railroad crossings and for river access areas. Recreational facilities also are provided. The federal construction project included modification to an existing pump station at a sewage treatment plant. The project included no new storm water pumping stations or ponding areas. Existing drainage facilities consisted of a combined sewer system serving about 5,160 acres. Sanitary sewage and storm water is handled at the wastewater treatment plant during floods, except for that which is pumped into the Mississippi River with portable pumps.

d. Muscatine. The project was authorized in 1961. Construction was completed in 1982. The levee and floodwalls were sized for coincidental 0.1% chance exceedance frequency events on Mad Creek and the Mississippi River. The project includes gated closure structures for several streets and roads, gatewells on drains and sewers that pass through the levees and walls, and a pumping station with a capacity 24,000 gpm, and a ponding area. The interior drainage area is about 172 acres. A warning system was recently installed for Mad Creek due to its flash flood nature.

e. Burlington. The construction project was authorized in 1977 under the continuing authority provided by Section 205 of the 1948 Flood Control Act. The project consists of about 1.5 miles of levees and floodwalls along Flint Creek and O'Connell Slough of the Mississippi River designed to protect 223 primarily industrial acres from floods up to the 0.5% frequency. The project includes a ponding area for temporary storage of runoff, gravity flow for discharge into the river at low stages, and a pumping station for use when gravity drainage is blocked. Gatewells for each gravity outlet prevent back flows at high river stage. Closure structures are provided for several street and railroad crossings. Other streets are ramped over the levee. A levee and drainage district was organized within the city of Burlington after the project was constructed. This provides a unique operational arrangement, with the industries providing most of the operation and maintenance rather than the City.

f. Des Moines. The project was authorized in 1944, and construction was initiated in 1966. The City of Des Moines is located along both banks of the Des Moines River between river miles 199 and 206 above its confluence with the Mississippi River. The City also extends for about 3 miles along the Raccoon River which confluences with the Des Moines River in the center of the City. The system of levees and floodwalls, with lengths totaling about 10 miles, extend along both banks of the Des Moines River and the Raccoon River. The project includes street and road ramps, sandbag closures, and a gate closure. Existing drainage facilities were modified, including the provision of gatewells, manholes, headwalls, flap gates, sluice gates, catch basins, relocations of existing pipes, and laying of new pipelines. The interior system includes no pumping plants.

The Des Moines local flood protection project is affected by two flood control reservoirs on the Des Moines River constructed and operated by the Corps of Engineers. Saylorville Dam is 9 miles upstream from the project. Red Rock Dam is 36 miles downstream. Saylorville Dam reduces the peak discharge on the Des Moines River but tends to increase the duration of high flows through the City of Des Moines. At full flood storage levels, Lake Red Rock can produce backwater effects on the Des Moines River up to and within the city limits of Des Moines. Remedial levees and a pumping station were constructed as part of the Red Rock project. These features are operated as part of the Red Rock Dam project with a physical tie to the lower end of the Des Moines local flood protection project.

g. Ottumwa. The levees, pumping stations, and sewers were designed and constructed by local interests, with construction being completed in 1963. The Corps of Engineers performed a study in 1968 that focused on interior flood protection. The resulting Corps construction project consisted of providing gated overflows and gatewells on an interceptor sewer to make it more efficient. The 12 feet by 10.5 feet concrete box interceptor sewer receives flows from an extensive combined sanitary and storm water collection system. The City of Ottumwa, population 35,000, straddles the Des Moines River about 50 miles downstream of Red Rock Dam. The dam has a significant impact on flood flows at Ottumwa.

h. South Quincy. The first construction contract for the recently completed federal project was awarded in 1987. The South Quincy Drainage and Levee District is located along the Illinois bank of the Mississippi River, immediately downstream of the City of Quincy. This is an old agricultural levee and drainage district that has become developed in part by industry. The Corps of Engineers improved the levees to provide protection for a 0.2% chance exceedance frequency flood, but could not justify improvements for the interior system. Local interests are in the process of designing a larger interior pumping station funded by a grant from the Economic Development Agency. The Corps' project consisted of increasing the height and strengthening 7.4 miles of existing mainstream sand levee, 1.4 miles of clay levee, and 0.4 mile of 3.5 feet high floodwall, providing gatewells and sumps in the industrial area, improving a railroad closure, and constructing a highway ramp. The South Quincy Drainage and Levee District is the local sponsor for the federal project.

i. Hannibal. Construction of this small local flood protection project was completed just a few weeks before the 1993 flood. The 0.7 mile of levee and floodwall is designed to protect the City of Hannibal against floods up to the 0.2% frequency event. The project includes four closure structures. Placement of the gate panels requires a crane that the city rents from a local contractor who has agreed to a 4-hour response time. The interior system includes a 40,000 gpm capacity pump station and five gatewells.

j. Performance During 1993 Flood. In general, the projects functioned as designed and were highly successful in preventing flood losses during the 1993 flood. The City of Des Moines suffered major damages due to (1) failure to close a railroad opening and (2) overtopping of a separate levee protecting the water treatment plant. There were no severe failures at the other projects. Several problems identified in the post-flood evaluation report (U.S. Army Corps of Engineers, Rock Island District 1995d) are noted in the following paragraphs. The performance of each project has some elements in common with those of other projects and some that are unique.

The most serious failure occurred in Des Moines. Due to inadequate warning time, the City was unable to make a sandbag closure of an opening for a railroad through the levee on the Raccoon River near the upstream end of the project. About three feet of water flowed through the opening and inundated an area of downtown. Lack of ponding and pumping facilities made the situation worst. Eventually, enough portable pumps were obtained to evacuate the flood water. Since the flood, a swinging gate has been installed for the railroad closure. However, city staff are concerned that warning time and manpower may be inadequate to sandbag many other street closures at bridges across the Des Moines River during future floods.

Flooding of the water treatment plant was another problem for Des Moines. The City's water treatment plant is located outside of the local flood protection project. The plant is protected by a separate levee, which was overtopped by the Raccoon River. The city was without potable water for several days.

The federal project in Des Moines included no pumping facilities. At the time the project was designed, pumping stations were considered a local responsibility. Interior flooding was a problem during the 1993 flood. Subsequent to the flood, the city is constructing a number of permanent pumping stations and also purchasing more portable pumps.

Problems with pumping plants, including power outages, capacity, and operating problems, occurred at several projects, with the nature of the problems differing from location to location. Additional portable pumps were needed at several projects.

A systematic problem was found at most projects with the mechanisms for sluice gate operation. Most operations personnel had trouble telling when the gates were fully closed. As a result, many had damaged some part of a mechanism or a gate stem due to over closing. Also, bent stems and rusty threads made operation of gatewells difficult.

Closure structures were noted for leaking badly at the side and bottom seals. Gates leaked even after being reinforced with sandbags. The rubber seals were generally considered to be too hard to form a good seal under low head. Sponge rubber was suggested by some operating personnel as being better in this regard. Portable pumps were used in several cases to return the leakage to the river.

Certain closures required more time to implement than indicated in the operations manuals. The closures were still made prior to flood stages reaching the sills.

Seepage through levees occurred at several projects. Levee seepage as well as leakage through closure structures may contribute to loads on interior systems. Seepage is a particularly significant source of flows to the interior system at the South Quincy project. The seepage for the peak river stage is roughly estimated to be about 28,000 gpm, as compared to a pump capacity of 60,000 gpm. The pumps were designed to handle interior runoff without consideration of seepage through the levee.

Unreliable flood forecasts and warning systems were noted by operating personnel at several projects as being a significant problem. Problems are primarily associated with smaller tributaries but also include forecasting stages on the Mississippi River. The Mill Creek early warning system for Milam did not work well due to false signals and unfamiliarity with the system. The Mad Creek warning system for the Muscatine project was found to be unreliable and vulnerable to lightning strikes. At the Burlington project, warnings for Flint Creek were considered to be four to six hours late. At some projects, forecasts for the Mississippi River were considered inadequate for optimal closing and opening of road closures and gatewells.

k. Lessons Learned. The U.S. Army Corps of Engineers, Rock Island District (1995d) evaluation of the performance during the 1993 flood of these ten urban levee projects resulted in the following conclusions and recommendations.

1. Information is lacking regarding the storm magnitude or frequency that the interior facilities are designed to handle. Local interests should be advised of the level of storm severity that will exceed system design capacities so that they can better assess their needs and risks.

2. In the analysis of interior systems, interior flooding is sometimes considered statistically independent of river stages. This may lead to underdesign. Further studies are needed in regard to coincident frequencies.

3. Dependability of warning systems, pumps, and gate operations decrease during intense storms due to power outages and operator error. Warning systems in general are not dependable and have been replaced by other means, including patrolling on a 24 hour basis during floods, use of telephone networking, and satellite weather reports. Improvements in early warning systems and forecasting are needed.

4. In some instances, designers have assumed that portable pumps would be used in time of need without providing guidance in regard to local interests obtaining pumps. Portable pumps should be given more attention, particularly where economic feasibility limits permanent pumping capacity.

5. Simplification is needed to help local interests understand the design concepts and train them for times of flood mobilization.

6. More attention needs to be given to emergency response times and personnel mobilization. Training and practice in making necessary closures, including locating and assembling components, are important.

7. Improved water seals are needed, especially on closure structures, stop logs, and panel closures.

8. Improved sluice gate operating mechanisms, stems, and indicators are needed to avoid damage during infrequent operation.

9. Ponding areas need to be made easier to maintain and should be under tighter control of the local authority.

10. The requirement for preparing a post-flood report was generally not met by the local sponsors. The data from this reporting procedure would be useful.

3-6. Huntington District's Project in Columbus, Ohio

The West Columbus local flood protection project was authorized by the Water Resources Development Act of 1990. Advanced engineering and design is underway. Several initial design memoranda were completed in 1991 and 1992. The Huntington District is responsible for design and construction. The city of Columbus is the local sponsor and has executed a Project Cooperative Agreement to cost share the design and construction of the project. Under this agreement, the city will assume operation and maintenance of the project following completion of construction. The Corps of Engineers will perform periodic inspections.

The project will provide protection for an area on the west side of the city of Columbus, Ohio, which is situated on low ground on the inside of a long meandering bend of the Scioto River. The 5.3 mile long levee and floodwall will protect about 1,300 acres. The protected area is primarily residential, with some commercial businesses, light industry, and public parks. The Scioto River has a drainage area of 1,629 square miles at a gaging station located two miles downstream of the project. The levee/floodwall provides protection for the standard project flood (SPF) with a minimum freeboard of two feet and three feet, respectively, provided at the initial overtopping section and at all other locations. The SPF was selected as the economically optimal level of protection.

a. Closures. The project includes closure facilities for 14 openings in the levee and floodwall. Nine of the closures are for streets and highways, four are for railroads, and one closes the opening for a pedestrian walkway. The closures are shown in Figure 3-6 and listed in Table 3-3 along with pertinent information including the type and size of closures and estimated closure times.

The estimates of personnel and time requirements, shown in Table 3-3, for operating the 14 closure facilities were developed in the Flood Warning Report, dated August 1993. Cumulative time of closure requirements are estimated for one, two, three, and four crews of eight persons each, to operate the 14 closure structures. The number of persons required to operate the closures will vary for any given flood event, depending on the forecasted crest stage and the available time to the peak. An estimated 37.9 hours is required for a crew of eight people to close the 14 openings. Four 8-man crews could complete the closures in an estimated 11.0 hours. The City of Columbus, Department of Public Utilities, Sewerage and Drainage Division will be responsible for developing individual operating plans for gate closures.

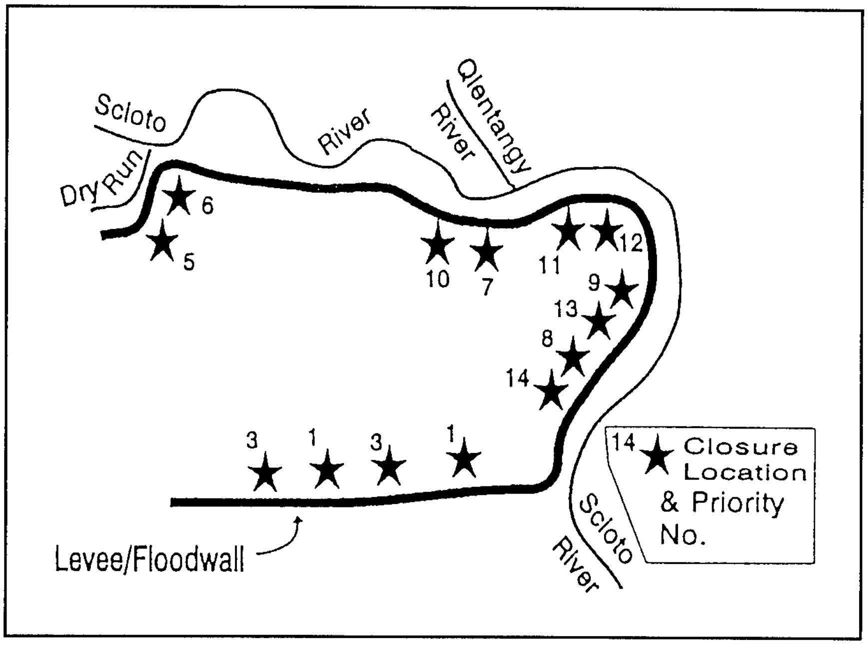


Figure 3-6. Local Protection Project in Columbus (U.S. Army Corps of Engineers, Huntington District 1994f)

Personnel will also be required, during floods, to monitor the perimeter of the project. An existing levee downstream of the project, that will affect the closure along Interstate Highway 70, will also be monitored. Closure of the project along the interstate highway may not be required for intermediate range floods on the Scioto River, depending on the integrity and stability of the existing downstream levee. Therefore, one person will be provided with communication equipment and assigned the responsibility of patrolling the downstream levee to observe any seepage distress, during forecasted floods in the 25-year to 75-year recurrence interval range. Meanwhile, others will be on stand-by status at a central location in case potentially dangerous conditions develop. This procedure will avoid any unnecessary closure and interruption of traffic patterns through the underpasses of I-70. Personnel will also be required to operate the interior flood protection system discussed next.

b. Interior System. The interior flood damage reduction system includes two storm water pumping stations with design capacities of 100,000 gpm and 180,000 gpm, respectively. Due to the high level of urban development and the topography of the protected area, provision of designated ponding areas is not feasible. The pumping stations will be supplied by existing storm sewer systems, that will be modified and expanded to facilitate collection and routing of flows to the pumping stations. One of the pumping stations will have a gravity outlet consisting of a 72-inch diameter circular conduit which bifurcates into two 54-inch conduits with flap gates at the outlet at the Scioto River. The other pumping station will have a gravity outlet consisting of two 7-ft by 8-ft rectangular box culverts that will connect to an existing storm sewer system. The capacities of the pumping stations and stormwater collection systems were selected based on traditional plan formulation and economic optimization. Residual flooding will occur whenever the capacity of the collector systems and pumping stations are exceeded.

Table 3-3
Closures for Columbus Project (U.S. Army Corps of Engineers, Huntington District 1994f)

Priority Closure No.	Location/Name	Closure Type[2]	Approx. Dimensions (WxH, ft.)	Stage at Gage	Sill Freq. (yrs)	Crew Size	Closure Time (hours) Number of Crews 1	2	3	4
1	Glenwood Ave.	QCG	50x17.9	25.1	20	3	1.2			
1	Souder Ave.	QCG	60x15.9	25.1	20	3	2.3	1.2		
3	Cypress Ave.	QCG	40x13.9	26.5	45	3	3.5	2.3	1.2	
3	Mound St./Ramp	QCG	36x13.9	26.5	45	3	4.7	3.5	2.3	1.2
5	McKinley Ave.	SL	34x8.8	26.7	50	8	7.0	4.7	3.5	3.5
6	Conrail RR(B)	SL	20x6.4	28.2	90	8	9.2	4.7	3.5	3.5
7	SR 315	QCG	132x10.2	28.5	100	8	10.4	5.8	3.6	3.5
8	Rich St.	SL	124x6.3	28.9	130	8	14.7	9.0	7.8	5.4
9	Town St.	SL	76x4.9	29.7	215	8	17.8	9.1	7.8	6.8
10	Souder Ave.	SL	50x5.4	30.1	285	8	20.1	11.3	7.8	6.8
11	CSX RR	SB	36x2.7	31.2	580	8	24.3	13.3	12.1	9.7
12	Conrail RR (c)	SB	36x2.2	31.1	520	8	29.5	16.4	12.1	9.7
13	Health Center	SB	32x1.4	31.4	660	8	32.8	16.6	12.1	10.1
14	CSX/Conrail RR	SB	72x1.4	31.6	740	8	37.9	21.7	15.3	11.0

[1] Beginning with departure from central location of crew assembly
[2] Closure Types: QCG = Quick Closure Gate, SL = Stop Log, SB = Sand Bag

The two stations will have similar arrangements for the pump sump, sluice gates, and gravity flow chamber. A sluice gate in the wet well will prevent back flow of pumped stormwater into the sump area during operation. The pump sump will be suppressed below the incoming storm sewer invert to accommodate submergence and cooling requirements for the submersible pumps. Therefore, a gravity flow chamber is provided to the side of the pump sump to pass gravity flow through the pump station during low flow river conditions. A sluice gate between the gravity flow chamber and the sump pump is provided to prevent gravity flow into the pump sump during interior storm events with low river conditions.

An overflow connection from a combined sanitary and storm sewer will allow overflows to pass through one of the pumping stations during high river conditions when the proposed sluice gate for the combined sewer is closed to prevent backflow from the river.

Personnel requirements, in additional to those noted previously for operating the closures, will include operation of the two storm water pump stations, numerous sluice gated structures, and sanitary sewer facilities. At least one person will be required at each of the two stormwater pump stations, while operation is required, with at least one other person rotating between the two for assistance.

The pump stations will be automated with respect to the cycling of the pumps, but human operators will be required to be on site during operation to rake the trash racks and ensure that the station is operating properly. The outfall sluice gate will be opened slightly, as necessary, to avoid an inordinately short cycle time of pump operation. After the completion of plans and specifications, the Huntington District will prepare an operation and maintenance manual that will include detailed procedures for operation of the proposed pump stations, storm sewer closures, and sanitary lift stations.

c. Flood Warning System. The Flood Warning Report for the West Columbus, Ohio Local Protection Project, dated August 1993, provides for an automated flood forecast system using computer workstations with communication interfaces to remote stream gages and rain gages located throughout the Scioto River drainage basin upstream of the project. The system will continuously collect and analyze data from the automatic reporting gages to provide real-time rainfall and stream level data for the Scioto River through the Columbus reach. Each gage will be provided with a threshold parameter, which will set off an alarm when met. At the alarm, city personnel will begin accessing and monitoring the warning system to keep abreast of developing conditions. At the same time, the National Weather Service will continue their normal procedure of providing bulletins and forecast stages for the Columbus reach of the Scioto River during the development of the impending storm event. The City of Columbus, National Weather Service, State of Ohio Emergency Management Agency, Franklin County Emergency Management Agency, and USACE Huntington District share responsibilities for implementing and maintaining the flood warning system. A Flood Alert Planning Committee will facilitate coordination of the warning system and evacuation programs.

The flood warning system will be linked with the State of Ohio Rain/Snow Monitoring System (STORMS). The STORMS system is a cooperative effort of state and local emergency management agencies (EMA) and the National Weather Service to develop an integrated flood warning system within state of Ohio that will enhance the ability of the NWS to assess flash flood potential. This program began a five year implementation period in 1993.

For the interim, the flood warning report provides a non-automated, or manual, method of forecasting to be used until project construction is complete, that will be retained as a backup for the automated system.

Many different combinations of meteorological conditions, rainfall amounts, and antecedent conditions can occur at any given time and river stage. Warning time may be highly variable. For example, as shown in Table 3-4, historic storms exceeding flood stage on the Scioto River at Columbus produced maximum 6-hour rates of rise ranging from 0.13 to 2.29 feet per hour. The Huntington District has performed extensive hydrologic and hydraulic analyses to develop the flood forecasting procedure for use in the interim awaiting implementation of the automated system.

Table 3-4
Rate of Rise During Historic Storms Scioto River at Columbus, Ohio
(U.S. Army Corps of Engineers, Huntington District 1994f)

Storm Event	Total Rainfall (inches)	Total Storm Rate of Rise (feet/hour)	Max 6-Hour W. S. Rise (feet)	Max 6-Hour Rate of Rise (feet/hour)
Mar 1913	6.97	0.58	7.09	1.18
Mar 1927	2.68	0.20	1.90	0.32
Mar 1933	5.00	0.27	6.16	1.03
Jan 1937	2.17	0.57	7.00	1.17
Jan 1952	4.00	0.54	7.10	1.18
Jan 1959	5.24	0.55	13.75	2.29
Mar 1963	3.02	0.24	1.65	0.28
Mar 1964	4.32	0.12	0.80	0.13
Feb 1975	2.45	0.32	3.10	0.52

Chapter 4
Performance, Function, and Workability

Performance, function, and workability of local flood protection projects refer to

- the performance of a local flood damage reduction project in reducing flood losses,
- the functioning of closure structures, interior facilities, and other components in contributing to overall project performance, and
- workability in terms of the responsibilities of organizations and people in implementing, maintaining, and operating the project to achieve the expected performance.

The performance of a local flood protection project in reducing flood losses is dependent on proper functioning of its components such as closure structures and interior facilities. Closure facilities, for highway, railroad, and other openings, prevent river flows from inundating interior areas. Gravity outlets and pumping stations discharge interior flows to the river and must also function properly to assure that river backflows are not inadvertently allowed to contribute to interior flows. Other components of the overall project play various roles in reducing flood losses.

4-1. Planning, Design, Maintenance, and Operation Considerations

Proper functioning, during floods, of closure structures and interior facilities in levee projects is dependent upon the effectiveness of the full sequence of:

- plan formulation and evaluation,
- preconstruction engineering and design,
- establishing institutional responsibilities and lines of communication,
- project implementation,
- maintenance of facilities,
- institutional capabilities and readiness,
- emergency preparedness planning, training, and exercises,
- implementation and operation of flood warning systems,
- mobilization of personnel and operation of closure structures and interior facilities during the flood event, and
- post-flood repair and evaluation activities.

Hydrologic, hydraulic, civil, geotechnical, structural, mechanical, electrical, and other engineering aspects of the design of closure structures and interior facilities, as well as other project components, are essential to project performance. The facilities must be designed to function effectively under a broad range of adverse conditions including operation by inexperienced personnel, quickly under urgent emergency conditions, during high winds and intense precipitation, after many years of less than optimal maintenance. Flood warning times, rate-of-rise, and consequences of less than optimal operation are major considerations in planning and designing the project.

Proper maintenance is also a key factor in achieving reliable project performance during floods. Local flood protection projects designed and constructed by the Corps of Engineers are maintained and operated by nonfederal sponsors, typically cities or levee or drainage districts. Several city departments or local agencies may be involved in project maintenance and operation. For example, for the project in St. Louis, the City of St. Louis Department of Streets is responsible for the levees, floodwalls, and closures, and the Metropolitan Sewer District is responsible for the pumping stations and interior system. The Corps performs periodic inspections, but mechanisms are lacking for effective enforcement of maintenance standards. Inadequate funding for infrastructure maintenance is widely recognized as being a significant problem throughout the United States.

Project performance depends upon personnel being mobilized and closures, gates, and pumping stations being operated in a timely and reliable manner. The reliability of the closure and interior facilities is dependent upon responsibilities being clearly defined and responsible parties being well prepared. A flood preparedness plan includes designation of organizations and individuals to be responsible for operation and maintenance of the various components of a local flood protection project. Closure structures also involve coordination with railroad companies, state transportation departments, and personnel responsible for traffic on city streets. Effective lines of communication must be established and maintained. Personnel must be trained and prepared to respond to infrequent emergency situations. Equipment and materials must be available when needed.

Timely and accurate information regarding levels and timing of river stages is crucial to project operations and other aspects of the emergency response to a flood. For watersheds characterized by rapidly rising streams or flash floods, warning and response time considerations may be the key element in determining functionality and performance. The reliability of project operations may be highly dependent on flood monitoring, forecasting, and warning systems.

Post-flood evaluations provide valuable lessons regarding strategies for improving the performance of the various components of local flood protection projects. Closure facilities, forecasting and warning systems, and interior flood damage reduction components may be modified to improve functionality. For example, physical facilities or maintenance and operation practices for most of the projects discussed in Chapter 3 were modified in some way in response to lessons learned during the 1993 flood. Pumping capacities were increased at several projects. A sandbag closure that failed to be closed during the flood due to time constraints was replaced with a gate that could be closed much quicker. Post-flood activities also include repair of damaged facilities.

4-2. Risks Associated with Local Flood Protection Projects

Reliability and risk are inherent in the concepts of performance, function, and workability. System components are evaluated from the perspective of their reliability, or conversely risk of failure, in functioning as intended in contributing to the performance objectives of the project. Risk management and analysis involve considerations of both the likelihood and consequences of failures of a facility to function as intended.

a. Types of Risk. Major flood losses may occur in the area protected by a levee project if one or more of the following conditions occur.

- The levee is overtopped by a flood event exceeding the capacity of the levee.
- Runoff from the interior watershed exceeds the capacity of one or more components of the interior flood damage reduction system.
- The levee or floodwall fails due to internal erosion, slope stability, structural problems, or overtopping by a flood less severe than the design flood.
- Closure structures do not function as intended.
- Interior facilities do not function as intended.

The first two conditions involve floods that exceed capacities. If a project is properly designed and operated, flood damages associated with an extreme event exceeding the capacity of the project do not imply a design or operational failure. The project simply does not provide complete protection against all floods. The third condition involves a levee failure. There is a risk that the levee or floodwall will fail to function as designed or intended. The problem could involve risks and uncertainties related to hydrologic, hydraulic, geotechnical, or structural considerations. The last two conditions involve failures of closure structures or interior facilities to function as intended, which is the primary focus of this document.

Thus, the risk of flood losses occurring are associated with two general types of situations.

- For a particular flood, all components of the local flood protection function as intended, and the project performs as expected. Damages occur because the flood is an extreme event that exceeds the capacity of the project.
- One or more components of the project fail to perform as designed or intended.

The discussion of levee design in Section 4-2(b) is representative of the first situation, in which management and analysis of risk are explicitly recognized in the planning, design, implementation, and operation of flood damage reduction projects. Likewise, storm sewers, channels, detention storage, and other components of interior flood damage reduction systems are designed with an explicit recognition of the risk of extreme flood events exceeding capacities.

The risk of failures of interior and closure facilities discussed in Section 4-3 is illustrative of the second situation noted above. Operating personnel may fail to close gates, or pumps may be inoperative due to power failures or other causes. In the past, risk of failure of closure and interior facilities has been neglected in quantitative analyses such as those outlined in Chapter 5. Experience shows that these facilities will not always function as designed and intended. The uncertainty is an important aspect of risk management and analysis. The objective of the remainder of Chapter 4 is to outline factors affecting the risk of interior and closure facilities not performing their functions as intended during a flood.

b. Levee Design. Local flood protection projects are formulated and designed based on maximizing net economic benefits consistent with acceptable risk and functional performance. Analyses are required to assure safe, reliable, and predictable performance of the project. The likelihood and consequences of future floods exceeding the project capacity are considered in evaluating the tradeoffs between risk, cost, and other factors. Overtopping by extreme events does not necessarily imply a design failure. A 0.5% annual exceedance frequency flood could naturally exceed the capacity of a levee, channel improvement, storm sewer, or reservoir that is properly sized for a 1% chance exceedance frequency event. Risks (estimated likelihoods and consequences) are evaluated; risk management is reflected in decision-making processes; and specified risks are explicitly accepted.

Economic considerations necessitate that projects be sized for less than complete protection. The capacity exceedance floods for the Corps of Engineers projects cited in Chapter 3 range from a 0.5% chance exceedance frequency at several projects to the standard project flood (SPF) for the project at Columbus. The level of protection may vary during the life of the project. When authorized in 1955, the St. Louis project was designed for an estimated 0.5% chance event. Due to construction of upstream reservoirs and availability of additional hydrologic information, this same river stage is now estimated to have a frequency of 0.2% to 0.1% chance exceedance. The level of protection provided by a project may also decrease as watershed development increases runoff potential. Erosion/sedimentation or river control projects may also change the discharge versus stage relationship.

The level of protection cited for the levee projects discussed in Chapter 3 reflect the past practice of including a minimum freeboard to compensate for uncertainties. In the past, in determining the top of levee profile, a minimum freeboard, typically three feet, has traditionally been added to the design water surface profile to account for hydraulic uncertainty. Under current policy outlined in ER 1105-2-101 (U.S. Army Corps of Engineers 1996a), the concept of freeboard is no longer used. As discussed in Chapter 5, under current policy, the reliability and performance of alternative levee heights are considered by explicitly incorporating the uncertainties associated with key variables in a risk-based analysis.

With either past or current evaluation methods, the risk of the levee being overtopped by events more severe than the design flood is recognized. The risks, consequences, and plans of action associated with floods exceeding project capacity are important considerations in plan formulation, evaluation, and design. Performance during events exceeding project capacity is considered as well as economic optimization. Flood preparedness plans include evacuation and other actions to be taken in the event of overtopping. Sandbags and other means have been used to increase levee heights during flood fighting efforts. Levee superiority refers to designing the top of levee profile such that, if capacity is exceeded, overtopping occurs at the least hazardous locations. Typically, overtopping in downstream reaches of a project are preferable to upstream reaches. Superior levee grades may be designed to prevent chain failures and a gradual extension of the length for design overtopping locations. Levee superiority may also be provided to protect pumping plants, water and wastewater treatment plants, and other crucial infrastructure should be protected.

4-3. Risk of Failure Associated with Closure Structures and Interior Facilities

There is a risk that one or more highway or railroad openings in a levee will not be closed as planned during a flood event. Likewise, flows from the river may inadvertently reach the interior through gravity drains or pumping station discharge lines. Flood damage from interior watershed runoff may result from operational mistakes, pump failures, and other problems with the interior flood damage reduction facilities.

A variety of factors could contribute to failure to close the openings. Personnel responsible for operating closure structures, gravity outlet gates, and pumping plants might be unable to accomplish their assigned tasks due to: insufficient warning time; inoperative facilities and equipment; electrical power failures during a severe storm; extreme weather conditions and impassable roads delaying access to facilities and equipment; communication problems; lack of preparedness for responding to infrequent emergencies; institutional failures in delineating responsibilities and providing resources; incompetence of individuals or organizations; or various combinations thereof. Structures and equipment may not function as designed due to poor maintenance, vandalism, or a variety of other reasons. Monitoring and warning systems and emergency response strategies may not be as effective as planned in the case of that particular actual flooding situation.

a. Closure Structures. The consequences of closure failure may range

- from the inconvenience of minor leaks through gate seals or stoplogs,
- to significant damages from major flows inundating interior areas,
- to catastrophic loss of life and property damage more severe than if the local flood protection project had not been constructed.

Failures to properly close openings during a flood could take various forms, such as the following.

- One or more openings may not be closed at all due to (1) inadequate warning time and/or response time, (2) inoperative facilities or equipment, (3) organizational deficiencies in delineating responsibilities, assigning and mobilizing personnel, and providing equipment and materials, or (4) some combination thereof.
- One or more openings may be only partially closed or closed after experiencing some flooding due to inadequate warning/response time, problems with the physical facilities, or inexperienced operations personnel.
- After being successfully closed, a closure structure may leak, inadvertently reopen, be overtopped, or fail structurally.
- A successfully closed closure structure may be later damaged by floating debris, extreme winds, automobile or train accidents, or vandalism.

Factors that affect the likelihood of closure failure can be categorized as follows:

- hydrologic and hydraulic characteristics of the river basin and associated flood characteristics,
- adverse weather conditions that may occur during a flood such as high winds, intense precipitation, hurricanes, or ice,
- effectiveness of flood monitoring, forecasting, and warning systems,
- configuration of the local flood protection project and number of closures,
- configuration and design of individual closure structures,
- traffic control operations that could affect timing of closures or the likelihood of accidents such as a train or automobile going through a closed gate,
- institutional, organizational, financial, and personnel capabilities for maintaining and operating the project, and
- perceived importance of the closure.

b. Interior Facilities. Systems of ponding areas, gravity outlets, and pumping plants function to discharge interior flows across the line of protection into the river. Interceptor channels or sewers may serve as an intermediate conveyor of flows from sewer and surface collector systems to the ponding, outlet, and pumping facilities. The overall interior flood damage reduction system may include stormwater management, drainage, combined sanitary and storm water collection, diversion, detention storage, channel improvement, and nonstructural measures located throughout the interior watershed. However, this document focuses specifically on ponding, gravity outlet, and pumping facilities for handling the interior flows at the line of protection.

As discussed in Chapter 3, interior flows reaching the line of protection are normally discharged through gravity outlets, whenever the river stage is below the interior water level, thus allowing gravity flow. Automatic flap-type gates or personnel operated slide gates prevent flow from the river from entering the interior when the river stage exceeds the interior water level. When the river blocks the gravity outlets, interior flood water is stored or pumped. There is a tradeoff between storage capacity and outlet discharge capacity. For a given level of flood protection, the size and costs of pumping stations and gravity outlets may be decreased with an increase in available detention ponding capacity. Detention storage also may increase the functional reliability of gravity outlets and pumping plants by providing additional time for personnel to mobilize and operate gates and pumps before damaging inundation depths are reached.

Detention ponding areas allow limited storage without significant damages occurring. As water is ponded to excessive depths above design levels, surrounding properties are inundated and damages occur. Backwater effects from excessive inundation depths at the line of protection may contribute to flooding throughout interior areas as sewers and channel capacities are exceeded. Many projects, including several cited in Table 3-1, have essentially no designated ponding storage. Damaging inundation depths occur whenever the inflows to the gravity outlets and pumping stations exceed capacities to discharge the flows across the line of protection.

Like other flood damage reduction measures, design capacities for ponding areas, gravity outlets, and pumping stations are limited by economics and other considerations. The present discussion is concerned primarily with failure of the facilities to function as designed and intended, rather than extreme flood events exceeding capacities. However, evaluation of interior facilities is complex, and capacity and functional failures are interrelated. Exceedance frequencies, for the capacities of the interior system and various components thereof, are difficult to quantify and may be essentially unknown. As noted in Chapter 3, information is severely lacking regarding the storm intensities and conditions required to exceed the capacities of the interior facilities of the case study local flood protection projects. Estimates of capacity exceedance probabilities for interior facilities are further complicated by watershed conditions that change over the life of the project. The reliability of pumping plants and gravity outlets, to function as intended in discharging interior flows, depends upon the accuracy of estimates of the exceedance probability and severity of floods that can be handled.

Failures for interior facilities to function as designed can be categorized as follows based on whether the problem involves normal interior flows or inadvertent backflows from the river:

- Runoff from interior watersheds, seepage, and other normal sources of interior waters are ponded in interior areas to depths that cause damages.
- Design or operational failures result in water from the river flowing backwards through outlet conduits across the line of protection to the interior.
- Leakage or seepage problems associated with conduits through the line of protection result in erosion or other types of damage to the levee, floodwall, and or appurtenant project structures. The source of eroding flows may be either interior flows or exterior river flows.

Most of the considerations noted earlier in Section 4-3(a), in regard to failure risks for closure structures, are also pertinent to interior facilities. Failures to properly close gates may result in flows from the river through either gravity outlet conduits or highway and railroad openings. However, failure risks for interior facilities are more complicated than for closure structures. There are more types of facilities that can fail in more ways.

Pumping stations are particularly vulnerable to the occurrence of unanticipated problems. The likelihood and consequences of failure for a pumping plant are affected by some factors that are unique to pumping plants and other factors that pertain to interior facilities in general. These factors include the following:

- the number of pumps or the proportion of the total pumping capacity that remains if one or two pumps are inoperative,
- the reliability of the power supply,
- type and design of pumps,
- configuration and design of the pumping station,
- configuration and capacity of the associated ponding area and gravity outlets,
- hydrologic and hydraulic characteristics of both the major river basin and the interior watershed,

- adverse weather conditions that may occur during a flood such as high winds, intense precipitation, hurricanes, or ice,
- effectiveness of flood monitoring, forecasting, and warning systems,
- institutional, organizational, financial, and personnel capabilities for maintaining and operating the project, and
- perceived importance of the pumping station.

4-4. Issues Identified in the Case Study Reviews

Chapter 3 summarizes reviews of several local flood protection projects, with a particular focus on their performance during the 1993 flood and lessons learned. Unlike numerous other primarily nonfederal agricultural levees that were overtopped, there were no levee overtoppings or catastrophic losses at the Corps of Engineers projects reviewed. The projects were highly successful in reducing flood losses. However, preventable damages and problems did occur that provide useful lessons. The 1993 flood experience motivated subsequent improvements in facilities, maintenance, and operations practices at all of the projects. Several key issues and concerns that surfaced during the reviews are cited below as examples of general problem areas that may adversely affect project performance.

- Examples of facilities not functioning as intended include the following.
    - Interior inundation occurred because a sandbag closure for a railroad opening along a tributary river was not made due to insufficient warning time.
    - A rusted-shut flap gate on an under-drainage system appears to be the cause of seepage erosion leading to severe damage of a floodwall, which potentially could have grown into a major breach.
    - Broken pipes entering a pumping station caused major erosion and subsidence.
    - Abandoned conduits through a levee allowed water from the river to flow to the interior.
    - Pumps were inoperative for several hours due to an electrical power failure.
    - Closure facilities were damaged during placement or required more time than anticipated.
    - Seepage under and through levees was common.
- In general, flood forecasting and warning systems were found to not be very reliable. For the several projects with local flood warning systems for smaller tributary watersheds, local authorities were generally dissatisfied with their systems. There was also some concern regarding the timing and reliability of National Weather Service stage forecasts for the Mississippi River.
- Inadequate pumping capacity was a significant problem at several projects. Use of portable pumps, in addition to permanent pumping stations, played an important role that may be overlooked in design studies.
- Maintenance is a major concern. Inadequate maintenance resulted in significant problems during the flood. No real mechanism exists for the Corps of Engineers to enforce nonfederal maintenance requirements.
- Evaluation of the level of protection provided by interior flood damage reduction facilities is highly uncertain and complex. Information is lacking regarding the level of protection provided by interior facilities at the existing projects.

- Design of the detailed features of facilities could be improved to solve the following types of common problems.
  - Seals on closure structures often leak especially under low head.
  - Mechanisms for closing gates are often damaged during emergency operations.
  - The amount of time required for closure operations could be reduced with better designs.
- Efficiencies in operation and maintenance may be achieved by incorporating features such as:
  - Automation of gate and pumping operations.
  - Mechanical trash raking equipment.
  - Use of submersible pumps in gravity outlets in lieu of pumping stations.
- From the perspective of basinwide flood response efforts, a better method is needed for transmitting current and accurate data in a timely manner regarding stages, flooded areas, precipitation, and other pertinent information. A geographic information system (GIS) could be useful in this regard.
- Since extreme flood events are infrequent, personnel are not experienced in operating projects during floods. Needs for more training and improved emergency preparedness were identified.

4-5. Consideration of Failure Risk in Planning and Design Studies

Risks of failures of closure and interior facilities to function as intended is a factor to be considered throughout planning, design, implementation, maintenance, and operation of local flood protection projects. During feasibility studies and advanced engineering and design, closure risk is a consideration in comparing tradeoffs between alternative plans and evaluating project feasibility. The likelihood and consequences of closure failure may be reflected in hydrology, hydraulics, and economic evaluation studies from various perspectives such as the following.

- Operation, maintenance, repair, and replacement costs for a local protection project during and following a flood event are much higher than during nonflood periods. Risks of closure and interior facilities failing to function perfectly as designed may be incorporated in project cost estimates for operations, maintenance, and replacements.
- Interior flood protection systems are normally designed primarily for handling storm runoff from interior watersheds. Treated sanitary sewage and seepage through and under levees also contribute to flows in interior systems. Likewise, leakage and flows through closures could be viewed as a potential contributing load on pumping plants and other components of an interior system.
- Flows through an opening may cause damage to otherwise protected properties and loss of life just like other river overflows or flows overtopping a levee. Likewise, failure of a pumping station may result in damages from interior flooding.

The analysis methods outlined in Chapter 5 focus on this last consideration, flows from the river and interior watershed damaging properties. The emphasis in Chapter 5 is on quantifying closure and interior facility failure risk from the perspective of incorporating probabilities and consequences of failures in the development of the frequency-damage, frequency-inundation depth, and related relationships, and average annual damage estimates that are normally developed during hydrology, hydraulics, and economics studies in support of the plan formulation and evaluation process.

Chapter 5
Risk-based Analysis Concepts

5-1. Risk and Uncertainty

Risk and uncertainty are inherent in all aspects of water resources planning and management. Risk implies a chance of loss, harm, or failure. Corps present policies for performing risk-based analysis for flood damage reduction studies are found in ER 1105-2-101, (U.S. Army Corps of Engineers 1996a). Technical procedures are covered under EM 1110-2-1619, (U.S. Army Corps of Engineers 1996b).

Traditional approaches for dealing with risk and uncertainty include: conservative estimates of modeling and evaluation parameters; safety factors, redundancy, and other forms of conservatism in designs; and analyses to test the sensitivity of decisions to estimates of key factors. In many cases, risks and uncertainties are simply ignored. During the early 1990's, risk-based analysis became a major emphasis in the Corps of Engineers. Risk-based analysis is an approach to evaluation and decision-making that explicitly, and to the extent practical, analytically incorporates considerations of risk and uncertainty. The objective is to facilitate more effective handling of risk and uncertainty in plan formulation and evaluation and decision-making processes. The effects of risk and uncertainty on the project design and economic viability are examined. Tradeoffs between risks, costs, and other considerations are explicitly incorporated in decisions.

5-2. Flood Damage Reduction Studies

*a. **Risk-based Analysis.*** ER 1105-2-101 (U.S. Army Corps of Engineers 1996a) states the Corps policy to adopt risk-based analysis procedures in flood damage reduction studies. In the risk-based analysis framework, key variables and parameters are treated probabilistically. For example, uncertainties are considered explicitly in developing discharge-exceedance probability, stage-discharge, and stage damage functions. These functions shown in Figure 5-1 are developed in the formulation and evaluation of flood damage reduction analyses.

Analyses of flood problems and flood damage reduction plans are complex. The relationships illustrated in Figure 5-1 represent a key element of a typical study approach. The floodplain is subdivided into reaches, and the functional relationships are developed for damage index locations representing each reach. The discharge-annual exceedance probability function is developed based on either (1) annual exceedance probability analysis of gaged streamflow data or (2) a watershed (precipitation-runoff) model. The discharge-stage relationship is developed using a water surface profile model and/or observed discharge-stage data. Economic studies are performed to develop the stage-damage functions. The basic functions are developed for each pertinent damage index location. Traditional analyses, combined the functions to produce probability damage function that was numerically integrated to obtain an expected value estimate of the annual damage.

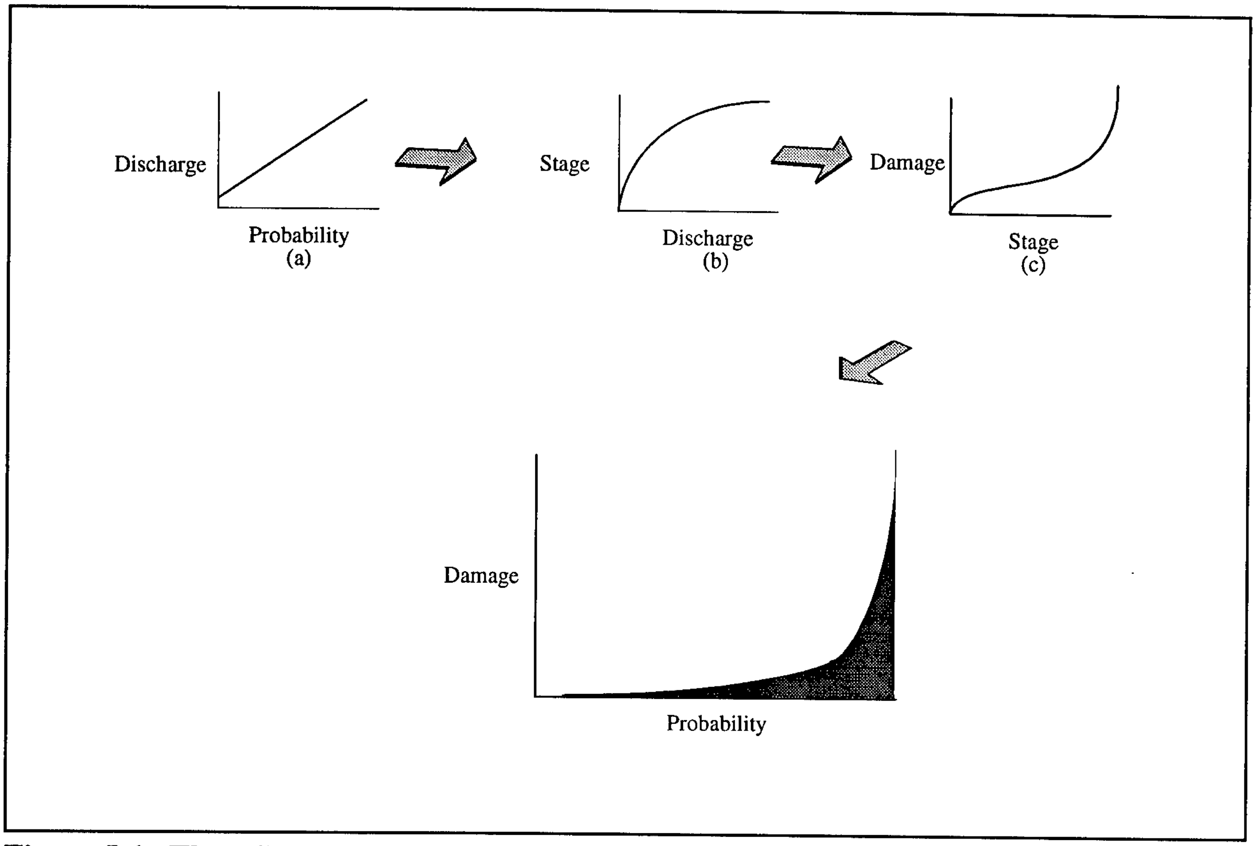


Figure 5-1. Three Basic Functions and Derived Probability-Damage Function

The risk-based analysis also uses the best estimates of the discharge-probability, stage-discharge, and damage-stage function. However, uncertainties, (errors) inherent in the basic functions are defined by probability distribution functions as shown in Figure 5-2. A Monte Carlo simulation is used to select a sample of each of the three functions. The selected functions are then combined and integrated as traditionally for a single simulation. The process is repeated thousands of times until a stable expected annual damage estimate is produced. The results are thus based on "all" possible combinations of the discharge, stage and damage functions considering the uncertainty of each. The procedures are described in EM 1110-2-1619 (U.S. Army Corps of Engineers 1996b).

The analysis is performed for without-project conditions and for each of the alternative plans being considered. An information base is developed for use in plan formulation and evaluation.

b. Effects of Modifications. Flood damage reduction measures are evaluated based on their impacts on the discharge, stage, and damage functions. Watershed management measures and storage projects alter the discharge-exceedance probability function for downstream locations. Channel modifications lower the stage for a given discharge. Nonstructural measures decrease the damage for a given stage. Levees prevent damages associated with a given river stage. Both levees and channel modifications can increase flow rates and stages downstream of the project due to increased hydraulic efficiencies and loss of overbank storage.

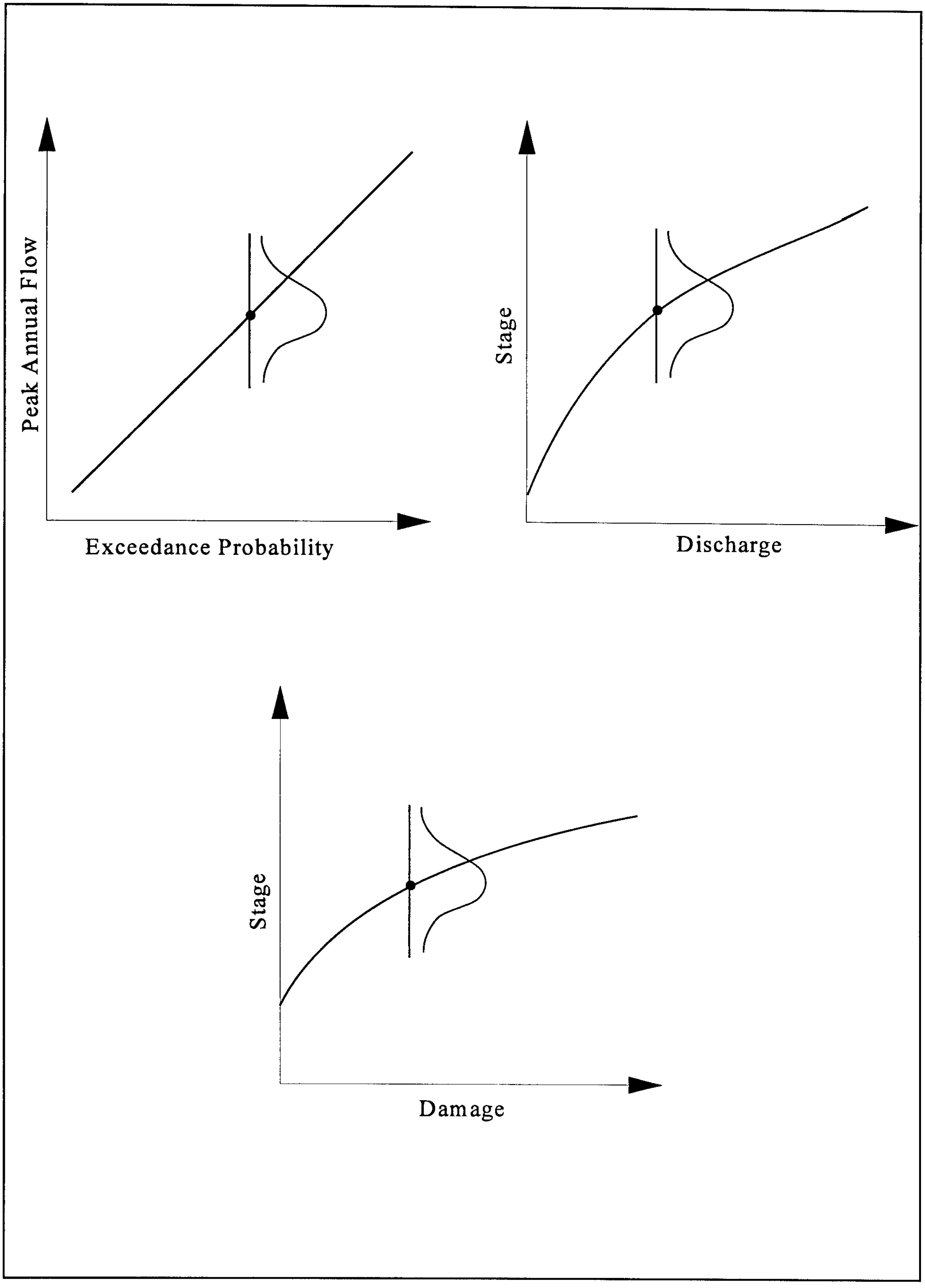


Figure 5-2. Uncertainty Reflected in Basic Relationships

Constriction of the flow, relative to without project conditions, may cause the stage to increase in the river adjacent and/or upstream of the levee. Floodplain properties are protected from river flows, and thus damages are prevented as long as the levee is not overtopped. Corps levees are designed so that capacity exceedance events will overtop in the least damaging manner. Normally, the overtopping is designed for the lower end of the levee.

c. Levee Capacity. Local flood protection projects are designed based on maximizing net economic benefits consistent with acceptable risk and functional performance. The risk of the levee being overtopped by more extreme floods is explicitly recognized during project formulation and in evaluation considering the tradeoffs between risk, costs, and other factors. Thus, overtopping by extreme events does not imply a design failure. However, the risk of exceeding the project capacity during its life is an important consideration in plan formulation and evaluation.

ER 1105-2-101 provides special guidance for levee and floodwall projects. The concept of freeboard to account for hydraulic uncertainty is no longer used. In the past, in determining the top of levee or floodwall profile, a minimum freeboard had traditionally been added to the design water surface profile. Under current policy, the performance of levee heights are considered by explicitly incorporating the uncertainties associated with key variables in a risk-based analysis. Past practice is continued in regard to designing the levee such that, if and when overtopping does occur, the overtopping is at a pre-planned least damaging location. Superiority is provided at pumping stations and other critical locations. Analyses are performed to assure safe, reliable, and predictable performance of the project.

5-3. Interior Flood Damage Reduction Systems

Flooding and stormwater management concerns in interior watersheds behind levees are essentially the same as in similar watersheds without levees, except for the complications caused by levees blocking the flow to the river. Interior flood damage reduction facilities can be categorized by whether or not they deal specifically with discharging interior flows through the line of protection. Structural and nonstructural measures at remote locations throughout the interior watershed reduce the flood damage in the same manner as similar measures in other watersheds where there are no levees. This document focuses on gravity outlets, pumping plants, and appurtenant facilities that deal specifically with discharging interior flows over or through levees and floodwalls. Flood damage reduction studies for interior watersheds incorporate the basic procedures previously described, including development of the functions illustrated by Figure 5-1.

a. Hydrologic Analyses. Guidelines for hydrologic analyses of interior areas are provided by EM 1110-2-1413 and ETL 1110-2-367 (U.S. Army Corps of Engineers 1987, 1995b). The hydrologic studies performed in planning and design of gravity outlets, pumping plants, and ponding areas include development of information characterizing the stage-exceedance probability of the ponding areas. Stage-exceedance probability, stage-duration functions, and related information are developed. Hydrologic analyses of interior facilities are complicated by fluctuating flows on both the exterior and interior sides of the line of protection. Water levels in the ponding areas are a function of inflows from the interior watershed and outflow discharged through or over the levee. Operation of gravity outlets and pumping plants depends upon river stages. For low river stages, interior flows are discharged through gravity outlets and pumping plants are not used. As the river rises, gravity outlet gates are closed and pumps are activated. The hydrologic analyses are normally performed using the continuous record or coincident frequency methods. The continuous record methods are characterized as:

- developing continuous river and interior flows for the period-of-record based on available streamflow and precipitation records combined with appropriate precipitation-runoff models,
- developing the river and interior flows for only pertinent flood events during the period-of-record based on available streamflow and precipitation records combined with appropriate precipitation-runoff models, or
- generating sequences of flows much longer than the period-of-record that preserve selected statistical characteristics of historical flows using techniques from stochastic hydrology.

Coincident frequency methods generate stage-frequency relationships for interior areas based on combining analyses as follows.

- A stage-duration relationship is developed for the pertinent location on the river using conventional hydrologic engineering methods. The full range of river stage is divided into n discrete increments, with each increment represented by a representative exterior stage (E_i) and probability ($P(E_i)$), where $\sum P(E_i)=1$.
- Flood hydrographs for several alternative exceedance frequencies are generated for the interior watershed using conventional hydrologic engineering methods. For each of the discrete exterior river stages (E_i), the interior runoff hydrographs are routed through the interior facilities to develop a stage-frequency ($P(I|E_i)$) function.
- The coincident interior elevation versus exceedance probability ($P(I)$) relationship is developed using the total probability theorem.

$$P(I) = \sum (P(I|E_i)P(E_i))$$

b. Minimum and Incremental Facilities Concepts. Plan formulation and evaluation are based on (1) treating "minimum" interior facilities as an integral part of the levee project and (2) incrementally considering additional interior facilities to reduce the residual flood damage. The evaluation of line-of-protection benefits reflect the minimum interior facilities. Further plan formulation and evaluation studies are performed to consider expansions to the interior flood damage reduction system. Interior facilities, in addition to the minimum facilities, must be incrementally justified.

The minimum interior facilities provide a level of interior flood protection representative of conditions without the levee project. The local runoff conveyance system should continue to function essentially as it does without the levee project, for floods up to the local storm sewer design capacity during periods of low exterior stages allowing gravity outlet flow conditions. Minimum interior facilities typically consist of natural detention storage and gravity outlets sized to meet the local drainage system. The minimum facilities may include other features, such as collector drains, excavated detention storage, and pumping plants if they are more cost effective.

Significant residual damages from interior flooding will typically still remain with the levee project, with minimum interior facilities, in place. Plans for additional interior facilities are formulated and evaluated separately. Reductions in expected annual damage and project costs for the full range of flood events are considered in the expansions to the interior flood damage reduction system. The additional facilities must be incrementally justified.

5-4. Failure Risk for Interior Facilities and Closure Structures

Flood damage occurs in the area protected if the capacity of either the levee or various components of the interior flood damage reduction system is exceeded. Even though all facilities function as designed, residual damage occurs when capacities are exceeded. Damages also may result from failures of one or more project components to function as designed and intended.

The remainder of this chapter focuses on failures associated with closure structures and interior facilities. The risk of damage occurring due to closure structures or systems of gravity outlets, pumping plants, and associated ponding areas failing to function as intended are addressed. Failures are categorized based on the source of the damaging flood waters, which is either (1) exterior flows from the river or (2) runoff from interior watersheds combined with seepage and other normal sources of interior flows. Section 5-5 deals with failures of pumping plants or gravity outlets that result in interior flows not being discharged over or through the levee as designed and intended. Section 5-6 addresses failures to block flows from the river from passing through the line of protection through highway and railroad openings, gravity outlet or pumping station discharge outlets, or other conduits or passageways through the levee.

5-5. Failure to Discharge Interior Flows

Pumping plants, gravity outlets, and ponding areas may fail to function as designed during a flood event. The failures may involve:

- less than optimal timing of gate and pump operations due to insufficient warning time, adverse weather conditions, inexperienced operations personnel, problems with facilities or equipment, or other reasons;
- failure of one or more pumps to operate during a portion of the flood due to pump malfunctioning or other problems;
- failure of one or more pumps to operate at all during a flood;
- failure of an entire pumping station to operate during a portion of the flood due to electrical power failure, operator error, inoperative equipment, or other problems;
- failure of an entire pumping station to operate at all during a flood;
- failure of one or more gravity outlets to function during a portion or all of a flood due to inoperative gates, operator error, inlets blocked by debris, or other problems;
- failure of a ponding area to provide the volume of storage intended prior to reaching damaging levels due to sedimentation, encroachment, or other constraints; and
- combinations thereof.

Ponding area detention storage increases reliability by providing additional time to mobilize and operate facilities before damaging water levels are reached, in addition to increasing the overall storage/discharge capacity of the gravity outlet, pumping plant, detention storage system. For projects without designated ponding areas, damage may begin to occur at minimal ponding depths.

a. Risk-based Analysis Framework. The risk-based analysis approach is applicable to evaluating the feasibility of flood damage reduction measures associated with interior areas. Development of the stage-exceedance probability function for the ponding area or at the inlet to gravity outlets and pumping stations is a key element in evaluating interior systems. The function and their associated definition of uncertainty are developed through hydrologic, hydraulic, and economic studies. Expected annual damage and a variety of other information are derived from the previous described Monte Carlo simulation process. This information is required to size gravity, outlets, pumping stations, and other components of the interior flood damage reduction system. The risk-based analysis approach is generally the same for the without- and with-project conditions considering the risk of the interior facilities failing to function as intended in disposing of the interior flood waters. Since the basic analysis procedures are required to evaluate interior facilities, incorporation of failure risk of the flood damage reduction measures has relatively little impact on the complexity or scope of a study.

Incorporation of failure risk, requires predictions of the probabilities and consequences of failures. The consequences of failures of interior facilities are reflected in the inundation depth at the levee and the backwater effects on water surface profiles throughout the interior area. Hydrologic and hydraulic models are developed to route inflow hydrographs through a system, consisting of a detention pond, gravity outlet, and/or pumping station, for a specified design and operating plan. Stage hydrographs for the ponding area or outlet facility inlet and water surface profiles through the interior channel and sewer system are thus determined for specified hydrologic flood conditions represented by interior hydrology and exterior river stages. The normal methods are then applied to relate stage to damage. A hypothetical postulated failure of a pumping station or outlet can be readily incorporated in the modeling.

The risk of failure of interior facilities is reflected in the probability description of the uncertainty associated with inundation depth or stage. For a given flood exceedance probability, the ponding area stage at the inlet to the gravity outlet or pumping station depends upon whether or not the facilities function as designed. A stage-exceedance probability function, as shown in Figure 5-3, is developed using conventional hydrologic engineering methods. The uncertainty associated with the function is estimated and includes various types of uncertainties including the risk of failure of the interior facilities. This uncertainty is illustrated in Figure 5-3 by defining the probability density function (PDF) for the stage-exceedance probability function throughout the range of the annual exceedance probability function.

b. Stage Uncertainties. The following sections outline a general framework for expressing risk associated with systems of gravity outlets, pumping plants, and ponding areas failing to handle interior flood water as intended. Failure of these interior facilities is reflected in the probability description of the uncertainty of the interior stage exceedance probability function.

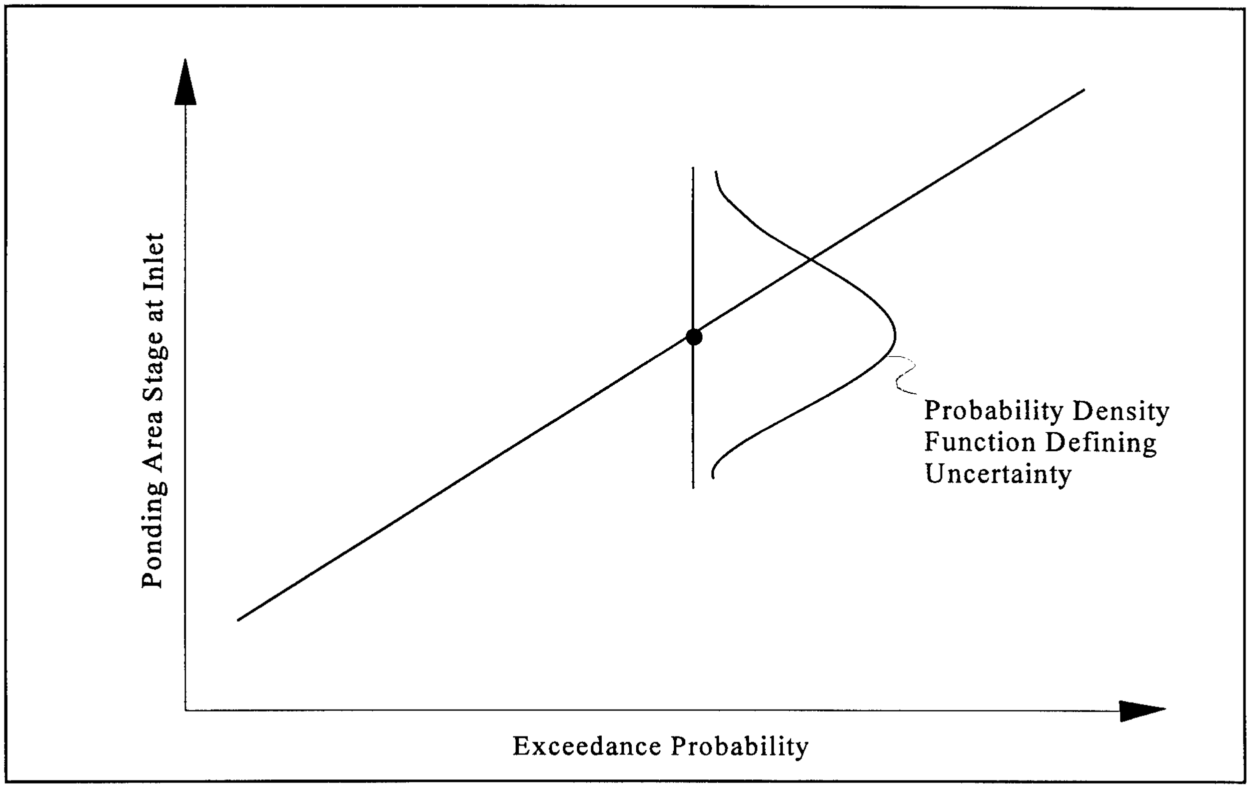


Figure 5-3. Interior Stage-Probability Relationship

Uncertainties associated with the stage hydrograph include (1) modeling uncertainties in developing relationships between exceedance probability, inflow, storage, outflow, and stage and (2) uncertainties regarding whether the facilities will function as intended. Various sources of uncertainty in the estimate of the interior stage associated with a specified exceedance probability include:

- uncertainties in the estimates of exterior river stages, reflecting hydrologic and hydraulic modeling premises, model parameters, gaged data, and premises regarding coincident flows;
- uncertainties in developing the runoff hydrograph from the interior watershed;
- uncertainties in estimates of interior flows from seepage and other sources;
- uncertainties in the models and parameters representing the hydraulics of the gravity outlets, pumping plant, ponding area;
- uncertainties in the elevation versus storage volume relationship characterizing the ponding area;
- uncertainties in the timing of the operation of the outlet gates and pumps related to warning and response times and other factors; and
- the risk that the outlets and pumping plant will fail to function as designed.

c. Uncertainty Probability Distribution Function. A general form for this probability distribution function of uncertainty for a single point along the stage-exceedance probability function is suggested in Figure 5-4. The random variable is the peak stage, depth, or water surface stage, in meters or feet, in the ponding area or, for projects with no designated ponding area, at the inlet of the gravity outlet and/or pumping plant, for specified flood flows of given magnitude represented by the exceedance probability of the flood event. The probability distribution of stage is shown in Figure 5-4 alternatively as a probability density function and as an exceedance probability function. An exceedance probability function is the integral of the density function. Piece-wise linear functions defined by four points provide a simple but realistic functional representation.

Potential failures, of a system for disposing of interior flood water, may be viewed as falling within the extremes of complete and no failure.

- No failure means that all facilities function as designed.
- Complete failure means that no interior water is discharged across the levee during the flood.

Likewise, when uncertainties related to modeling premises and parameters as well as functioning of the interior facilities are considered, the stage may be viewed as falling within a range defined by minimum and maximum values. These two extremes and two other intermediate points are used to define the probability function illustrated in Figure 5-4. Significant flexibility exists for adapting this general strategy to fit the scope of a particular feasibility study.

The following four points may be adopted to define the probability distribution at a specific point (probabilistic) on the stage-exceedance probability function. The process would be repeated for several point or events.

1. The point labeled 1 in Figure 5-4 represents the most likely scenario. The interior facilities function as designed. Most likely values are assigned to modeling parameters.
2. The maximum stage, point 2, represents the worst possible case scenario which includes discharge of none of the interior flow through the line of protection. All inflows from the interior watershed pond behind the levee. This would normally represent a small probability of occurrence.
3. The minimum stage, point 3, represents the best case scenario. The interior facilities function as designed. Values are assigned to modeling parameters that minimize stage but are still reasonable. The value might be considered as two standard deviations from the most likely condition value.

4. Point 4 represents a conservative scenario, in which the interior facilities discharge water through the line of protection at less than their design capacity, and model parameters are set to result in a conservatively high stage. The value might be considered as two standard deviations from the most likely condition value.

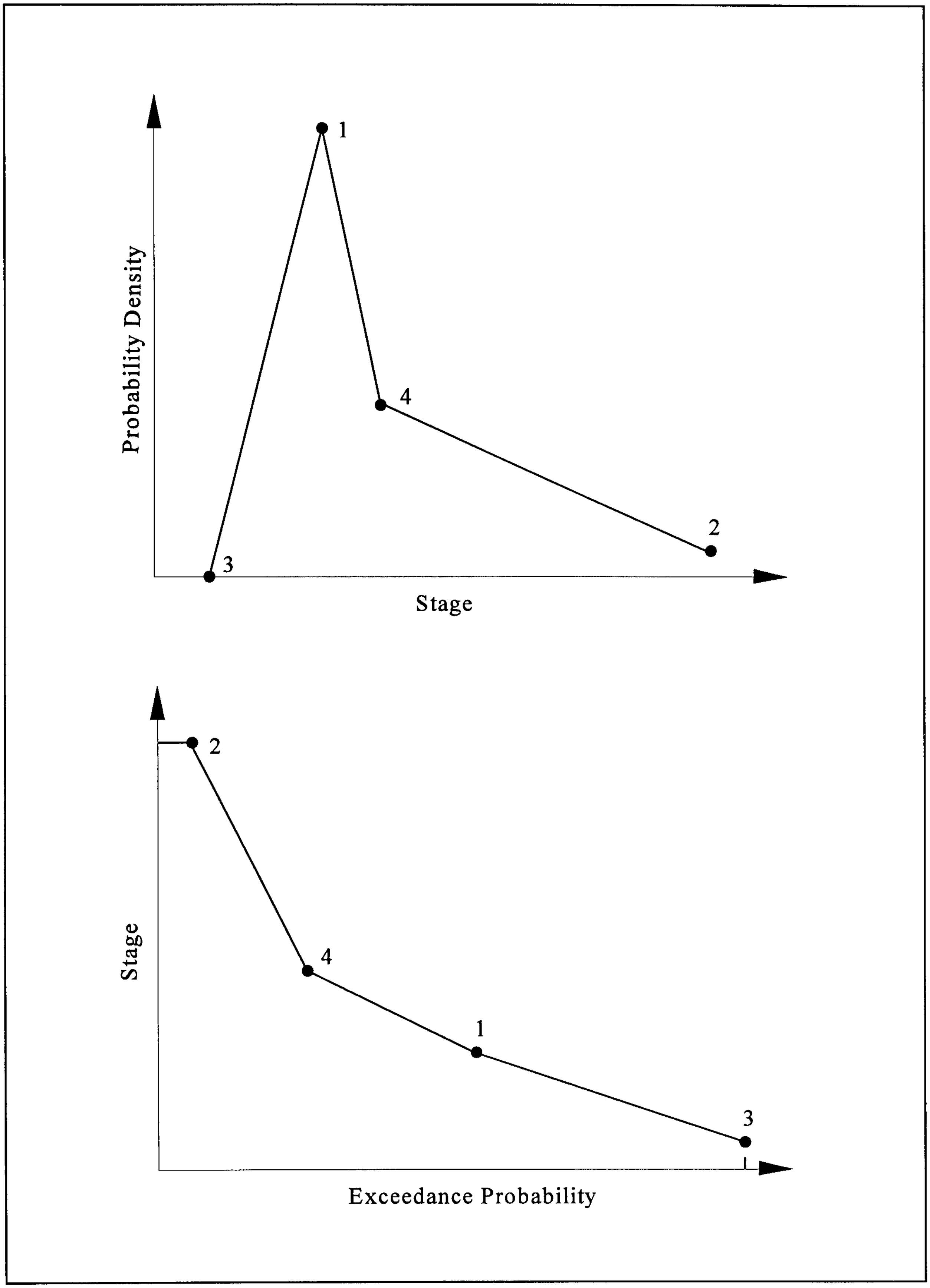


Figure 5-4. Probability Distribution Function Representing Interior Stage Uncertainty

Point 4 is added to derive a reasonable shape for the distribution function. There is some small probability that no flows will be discharged through the levee during the duration of the flood event (point 2). There is a typically much greater probability that interior stages will significantly exceed expected levels for that particular magnitude of flood but still be lower than the worst case of no outflow at all.

Judgement is required to formulate scenarios representing the four points that will realistically define the shape of the probability distribution function for a specific probability. For either the most likely scenario, the extremes, or the intermediate conservative condition, conventional hydrologic engineering methods may be applied to determine inundation stages. Estimates of the exceedance probabilities for the four points must be largely subjective, based on professional judgement, and are necessarily approximate. However, the probability estimates should reflect a thorough understanding of project features and flood characteristics developed through various analyses. Factors affecting the risk of failure are outlined in Chapter 4. Considerations in developing probability estimates are discussed in Section 5-7 from the perspective of closure risk but are also pertinent to failure to dispose of interior flood water.

5-6. Failure to Block River Flows

There is also a risk that openings in a levee or floodwall may not be closed during a flood as planned. Exterior flows would cause damage in otherwise protected interior areas. The openings could be gravity outlets or discharge lines from pumping plants that extend through the levee. The gates provided in these interior flow outlet conduits, to prevent backflows from exterior, may fail to function as designed due to physical circumstances and/or human operator error. Likewise, the closure structures for openings for highways, streets, railroads, and pedestrian walkways may not function as intended.

Risks are associated with:

- failure of the closure to be made at all during the flood,
- failure of the closure to be made in sufficient time, resulting in significant flows through the opening before closure is completed, and
- failure of the closure, after being made, to provide complete protection throughout the flood, ranging from nuisance leaks to major flows to catastrophic breaching.

The consequences of failure can be viewed in terms of:

- hydraulics of flow rates, velocities, and depths as the flood waters from the river flow through the opening and inundate interior areas,
- adverse impacts on functioning of other system components, such as detriments to emergency operations, overloading of interior flood protection systems, and erosion that could lead to breaching of the levee, and
- economic and noneconomic damage to people and property.

In the following discussion, the consequences of failure are addressed from the perspective of inundation depths and economic damages.

a. Failure/Nonfailure Approach. The least complicated approach for incorporating closure risk in the overall risk-based analysis process is to adopt the following premises:

- the closure is either a complete success or a complete failure,
- the failure probability is constant for exterior stages between the bottom of opening and top of levee, and
- the floodplain stage is inundated to the exterior stage if a failure occurs.

With this approach, the only additional information required to incorporate levee closure risk is an estimate of the probability of closure failure ($P(F)_S$) as a function of river stage. With the probability of failure assumed constant for the full range of river stage between the bottom of the opening and top of levee, the subscript "s" is an index of whether or not the exterior stage falls in this range. $P(F)_S$ is zero for exterior stages falling below the bottom of the opening and undefined for stages above the top of levee. Degrees of failure and corresponding levels of consequences are not differentiated. This approach may be adequate for some studies. In other cases, the alternative more detailed approach outlined in Section 5-6(b) will be warranted.

If only the probability of nonfailure, $P(NF)_S$, and probability of failure, $P(F)_S$, are considered, the reliability and risk probabilities are related as follows for a specified river stage denoted by subscripts.

$$P(F)_S + P(NF)_S = 1$$

The failure probability is expressed as the following discrete probability distribution.

$P(F)_S = p$ if exterior stage falls within the specified range
$P(F)_S = 0$ otherwise

where p is an estimate of the probability of the opening not being properly closed during the flood. Likewise, the reliability or probability of the opening being properly closed during a flood event is expressed by the following nonfailure (NF) probability distribution function.

$P(NF) = 1 - p$ if the exterior stage fall outside the specified range
$P(NF) = 0$ otherwise

For stages above the bottom elevation of the opening, the economic consequences of closure failure are conservatively approximated by assuming the water surface in the interior floodplain is the same as in the exterior. The exterior stage is applied to the stage-damage relationship to determine damages, similarly to the previously discussed approach illustrated in Figure 5-5 for representing levee overtopping. In actuality, the inundation depth will likely be less than the exterior. An alternative approach is to establish an interior stage for a specified exterior hydrograph and levee openings using hydraulic modeling. The computed interior stage is then applied to the stage-damage relationship to determine the damages associated with the failure probability.

b. Exterior-interior Stage Relationship. A more detailed evaluation of the closure risk may be warranted in many cases. The analysis is complicated by the variable relationship between failure probability, failure consequences, and exterior stage. A closure failure could allow a significant but limited amount of interior flooding involving stages that are significantly lower than exterior stages. Flow through an unblocked or partially blocked opening may be of varying duration, depth, and flow rate depending on a variety of circumstances. Multiple openings also complicate the analysis. A failure to properly close one or several openings may occur simultaneously with successful closures of other openings in the local flood protection project. The probability of a partial failure to properly close one or more openings may be much higher than for a complete failure.

Figure 5-5 compares the interior inundation stage to result from closure failure for (a) the case of the maximum water surface elevation in the interior reaching the maximum and (b) a less severe case of limited flow through the opening.

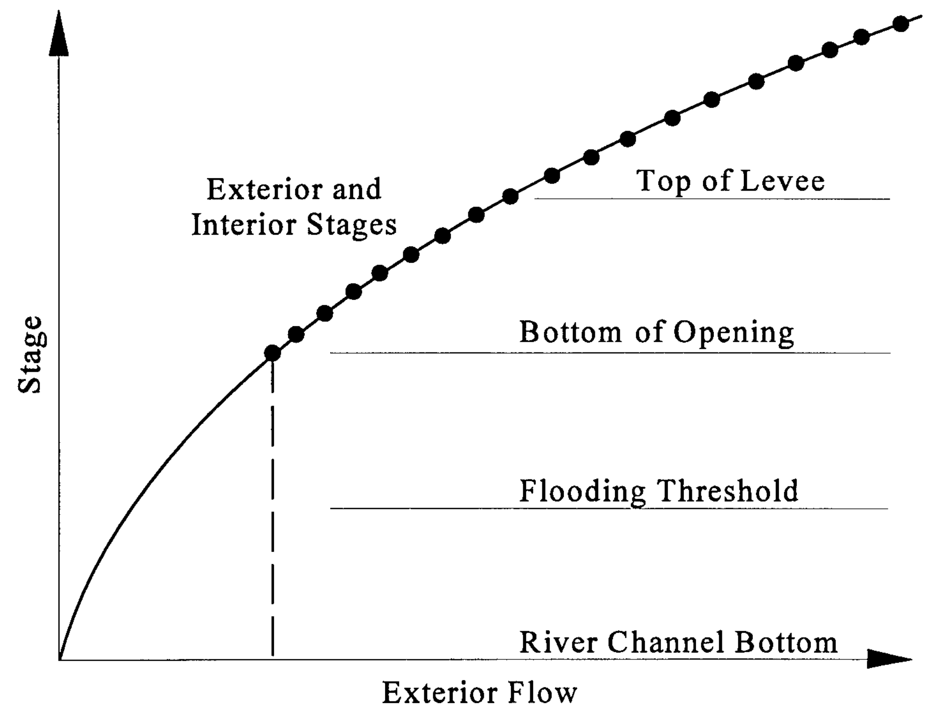
Detailed hydrologic and hydraulic analyses are required to delineate inundation areas for the second case of a postulated partial closure failure.

Developing the interior stage-exceedance probability functions for the range of exterior stages, may be organized based on considering the two extremes of no damage and worst case damage along with intermediate levels of flooding severity.

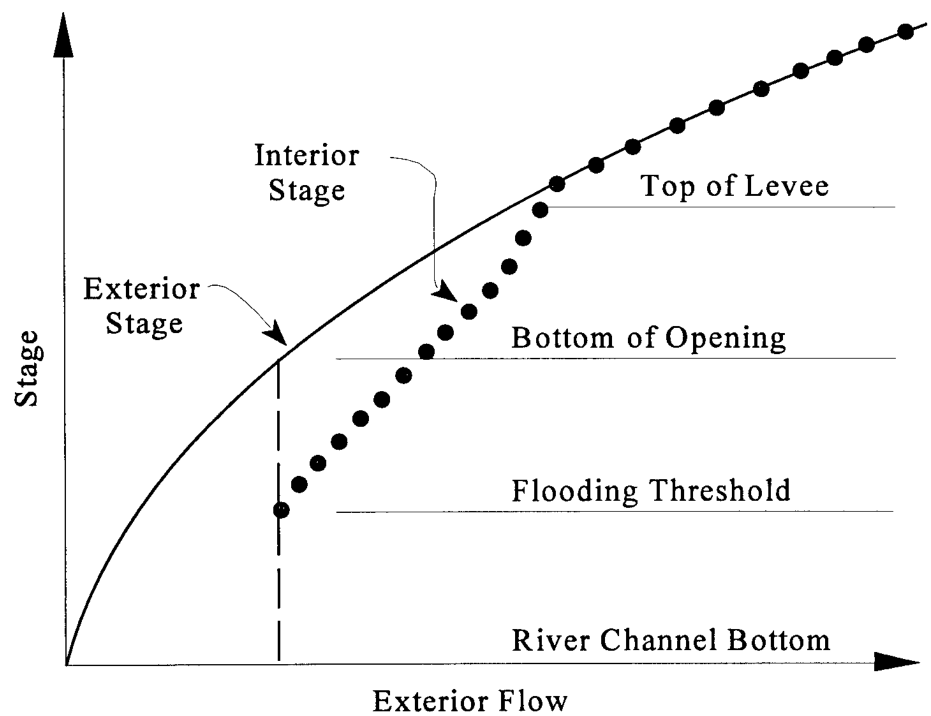
- The no damage (ND) scenario is defined as all closures being completed such that the interior floodplain inundation stage is less than a specified threshold required for damage to occur. Either the closure of all openings is completely successful or at least successful to the extent that any flows through the openings are too small to cause significant inundation.
- The worst case or complete failure (CF) scenario consists of flow through one or more openings inundating the interior area to a specified maximum stage, which is equal to or less than the exterior stage.
- Partial failure (PF_1, PF_2, PF_3, ..., PF_N) scenarios result in specified levels of interior inundation that cause significant damage but are less than the worst case scenario.

Estimates of the probabilities $P(ND|S)$ and $P(CF|S)$, conditioned upon specified exterior stage (S), associated with the nondamaging (ND) and complete failure (CF) scenarios define the extremes of the probability curves. Partial failure scenarios are formulated and evaluated as necessary to define the shape of the probability curves. The conditional probabilities, $P(PF_1|S)$, $P(PF_2|S)$, ... , $P(PF_N|S)$, associated with the N partial failure scenarios are expressed in terms of the probability that the interior inundation stage equals or exceeds a specified level, for a given exterior stage.

c. Example. A hypothetical example is used to illustrate the general form of the probability distribution relationship representing closure risk. A hypothetical small local flood protection project includes two levee openings, for a railroad and a highway. Pertinent elevations at an index location are as follows.



a. Interior and Exterior Stages Equal



b. Interior Stages Less Than Exterior Stages.

Figure 5-5. Exterior and Interior Stage Relationships

exterior channel bottom	-0-
threshold damaging depth of interior inundation	15 feet
bottom of line of protection for railroad opening through	21 feet
bottom of line of protection for highway opening through	24 feet
top of line of protection (levee)	30 feet

Exceedance probability versus floodplain stage curves are developed for exterior stages of 23, 26, and 30 feet. Probabilities of closure failure are zero for exterior stages of 21 feet and below. For exterior stages above 30 feet, the levee will be overtopped, the floodplain stage is assumed equal to the exterior stage, and the failure probability is assumed to be 100%. Since flood fighting efforts might include raising the levee and closure with sandbags, the failure probability for a exterior stage of 30 feet might actually be less than 100%, but 100% will be used as a conservative estimate for this example.

The following closure failure scenarios are formulated and evaluated to develop probability estimates.

non-damage (ND) exceedance scenario:	floodplain stage $\geq$ 15 feet
partial failure 1 (PF_1) exceedance scenario:	floodplain stage $\geq$ 18 feet
complete failure (CF) scenario:	floodplain stage = exterior stage

The following estimates are made of the conditional (for specified river stage) nonfailure (NF) probabilities that the closures are successful such that the damage threshold stage of 15 feet in the flood plain is not exceeded.

$P(NF|S=23) = 92\%$ $\qquad$ $P(NF|S=26) = 88\%$ $\qquad$ $P(NF|S=30) = 80\%$

Thus, for a exterior stage of 23 feet, the estimated probability is 92% that the floodplain stage will stay below the damage threshold of 15 feet. This means there is an 8% (100% less 92%) chance that a closure failure will result in the floodplain stage exceeding 15 feet, if the exterior rises to a maximum stage of 23 feet. The probabilities of the threshold stage not being exceeded are converted to the following non-damage (ND) exceedance probabilities.

$P(ND|S=23) = 8\%$ $\qquad$ $P(ND|S=26) = 12\%$ $\qquad$ $P(ND|S=30) = 20\%$

The exceedance probabilities for complete and partial closure failures are estimated as follows.

$P(CF|S=23) = 2$ $\qquad$ $P(CF|S=26) = 3\%$ $\qquad$ $P(CF|S=30) = 4\%$

$P(PF_1|S=23) = 4\%$ $\qquad$ $P(PF_1|S=26) = 6\%$ $\qquad$ $P(PF_1|S=30) = 10\%$

The probability distributions can be visualized graphically as illustrated in Figure 5-6. For lack of better information, the functions are assumed linear between the points estimated for the defined scenarios. Linear interpolation may also be used to determine the frequency-floodplain stage relationship for other exterior stages.

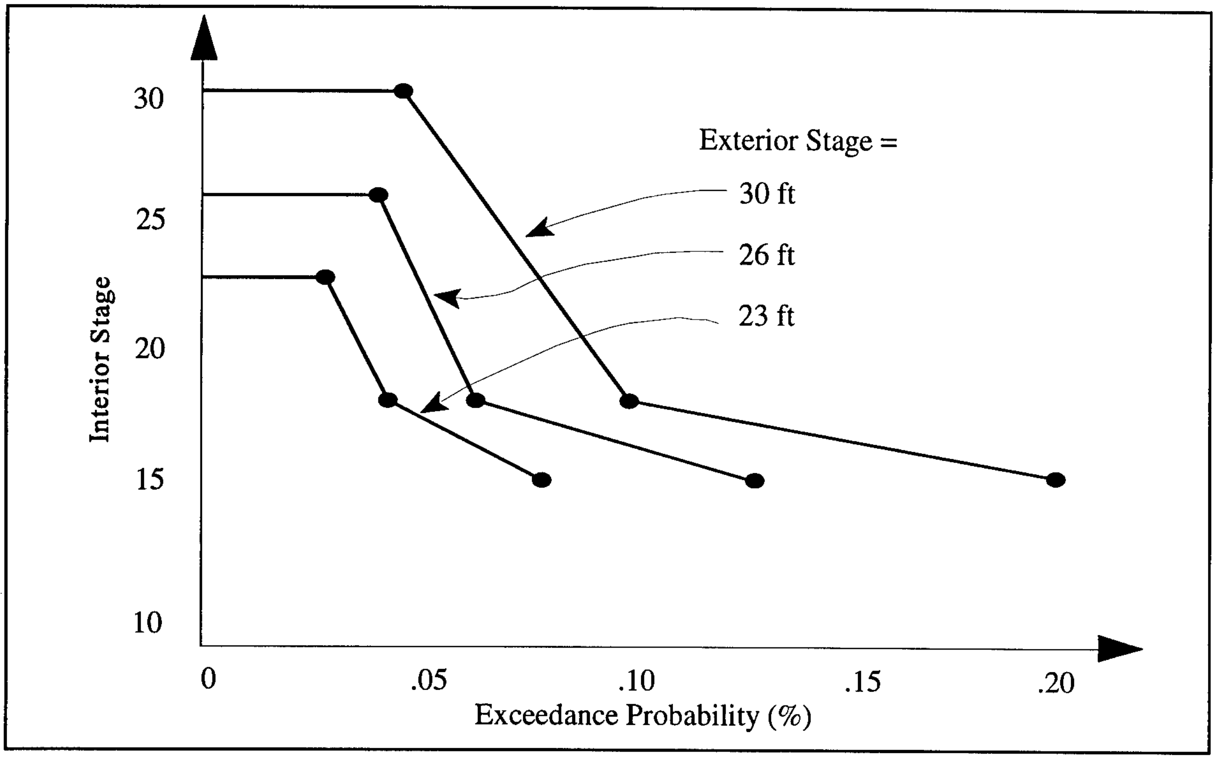


Figure 5-6. Example Interior Stage-Exceedance Probability Function

d. Incorporation of Closure Risk into the Overall Analysis Framework. The exterior stage - floodplain stage - exceedance probability relationships, representing closure risk, may be incorporated into the overall risk-based analysis approach, outlined in Section 5-2, in a variety of ways. Two alternative approaches are described in the following Sections 5-6(d)&(e). Variations of these two general approaches can be adapted to the scope of a particular study. The two alternative computational strategies are:

- to directly combine the probability-stage relationships, representing closure risk, with the other basic relationships to derive expected annual damages and other pertinent information, and
- to incorporate the closure failure probability relationships directly into the Monte Carlo simulation.

With the first approach, the conditional expected value of damage and expected annual damage associated with closure failures, as well as other pertinent information, are developed outside of the Monte Carlo simulations. The conditional (conditioned on river stage) expected value of closure failure damages may be added to the stage-damage functions provided as input to the Monte Carlo simulations. In the second general approach, closure risk is directly represented in the Monte Carlo simulations either by the failure probability-stage relationships of Sections 5-6(b)&(c) or the failure/nonfailure probabilities of Section 5-6(a).

e. Direct Combining of Functional Relations. For a given exterior stage, the conditional exceedance probability versus floodplain stage relationships illustrated by Figure 5-6 may be combined with the stage versus damage relationship to develop a relationship between exceedance probability and damage, for a given exterior stage, which can then be numerically integrated to obtain the expected value of damages. Thus, expected or average damage, associated with closure failure, may be obtained for each exterior stage. These are the conditional expected damages for specified river stages. The expected damage, conditioned on river stage, may be combined with the annual exceedance probability versus exterior stage relationship to obtain the unconditional annual exceedance probability versus damage relationship. This relationship is numerically integrated to obtain the expected or average annual damages associated with closure failure. The expected annual closure failure damages computed in this manner may be treated as a component of the overall residual expected annual damage for the project.

Likewise, the conditional exceedance probability versus floodplain stage relationships, representing closure risk, can be numerically integrated to obtain the expected value of the floodplain stage for each river stage. This information may be combined with the annual exceedance probability versus river stage relationship to derive a relationship between annual exceedance probability and floodplain stage.

A closure failure probability relationship may also be derived. Closure failure is defined as the interior stage exceeding the damage threshold (or some other specified level). The conditional failure probabilities for each exterior stage are combined with the exterior stage-annual exceedance function to compute an unconditional or total failure probability.

The conditional expected value of closure failure damages, for specified river stages, may be added to the stage-damage functions provided as input to the Monte Carlo simulations, with no changes to the steps outlined in Section 5-2(b). Alternatively, as discussed next, closure risk may be included directly by sampling during the Monte Carlo simulation.

f. Monte Carlo Simulation. As discussed in Section 5-2(b), the relationships illustrated in Figure 5-2 are combined using Monte Carlo simulation. Many thousands of Monte Carlo experiments are performed, with each representing an annual flood event. The experiment to simulate each annual flood consists of the steps outlined in Section 5-2(b). This list of steps is repeated below with closure risk included.

1. The discharge-exceedance probability function is sampled to determine a peak flood flow, to which a random error is added by sampling.
2. The exterior stage corresponding to the flow is determined from the discharge-stage relationship, and a exterior stage prediction error is added by sampling.
3. The floodplain stage corresponding to the river stage is determined by sampling either

    - the failure/nonfailure probability distribution described in Section 5-6(a) or
    - the frequency-floodplain stage relationship, representing the risk of closure failure, described in Section 5-6(b).

4. The damage corresponding to the floodplain stage is determined from the stage-damage relationship, to which a random error is added by sampling.

The Monte Carlo sampling in step 3, associated with closure risk, is performed in the same manner as the sampling in the other steps. The failure/nonfailure approach of Section 5-6(a) and the frequency-floodplain stage relationship approach of Section 5-6(b) are similarly incorporated in the Monte Carlo simulation. A random number is selected from the standard uniform distribution. With the failure/nonfailure approach, the floodplain stage is assumed to be either equal to the river stage or nondamaging, depending on whether or not the sampled random number is greater than the failure probability $P(F)_S$. If the approach of developing a frequency-floodplain stage relationship is adopted, the exceedance probability is set equal to the random number drawn in the Monte Carlo sampling. The exceedance frequency is applied to the frequency versus floodplain stage relationship, reflecting closure risk, to determine the floodplain stage.

Many thousands of annual flood events are simulated by repeating steps 1 through 4. Average annual damages are computed by averaging the thousands of annual damages. A closure failure frequency relationship may be developed from the results of step 3 by counting the number of experiments for which a closure failure occurs or the floodplain stage exceeded the specified nondamaging level.

5-7. Considerations in Developing Closure Risk Estimates

River stage - floodplain stage - exceedance probability relationships provide a mechanism to incorporate risk of closure failure into the risk-based analysis framework described in Section 5-2. Incorporation of these relationships into the overall risk-based analysis framework is outlined in Section 5-6. The remainder of Chapter 5 focuses on development of the relationships. Although the focus of the following Section 5-7 is on closures, many of the basic concepts expressed are pertinent to the failure risks associated with interior flows discussed Section 5-5 as well.

a. Basic Relationships. The relationships to be developed are illustrated in Figure 5-7. A depth of inundation (stage) versus exceedance probability relationship, at a damage index location in the protected interior floodplain, is developed for a given river stage. Floodplain stage versus exceedance probability relationships are developed for several river stages covering the full range from the bottom of the lowest opening to the top of the levee. Relationships may be developed for each of several damage index locations representing different reaches or areas of the floodplain.

For the simplest failure/nonfailure analysis approach described in Section 5-6(a), the only failure probability used is that corresponding to the maximum floodplain stage which is assumed equal to the river stage. The failure probability is assumed constant for all river stages between the bottom of the lowest opening and the top of levee. The basic concepts discussed in the present Section 5-7 are generally pertinent to estimation of a single failure probability as well as to development of the complete relationships shown in Figure 5-7. The relationships of Figure 5-7 are not needed for an analysis based on the simplifying assumptions of Section 5-6(a).

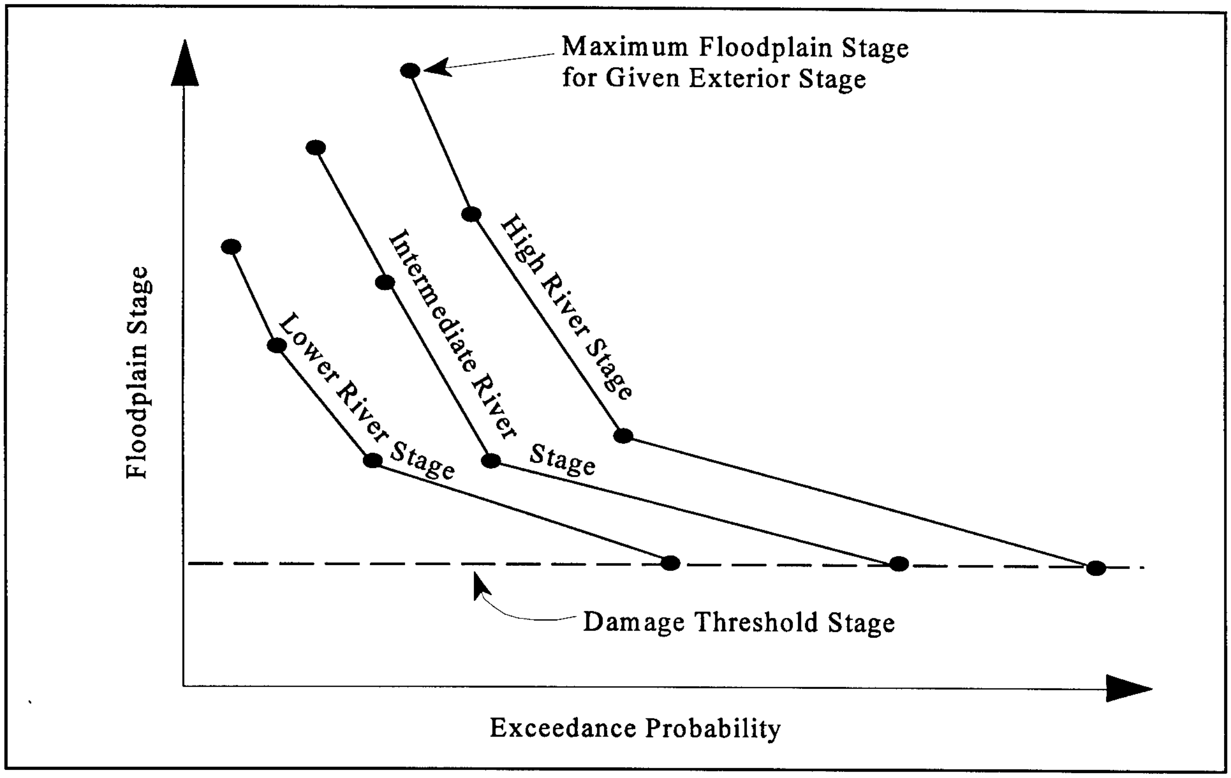


Figure 5-7. Frequency-Stage Curves for Closure Risk

However, as discussed in Section 5-6(b), the relationships are required for more detailed analyses. The format of the probability versus stage relationships can be varied in various ways to fit the level of detail and needs of a particular feasibility study.

The general framework for defining the basic relationships consists of the following premises.

- A closure failure is represented as a relation between river stage and floodplain stage. For a given river stage, closure failures are defined in terms of floodplain stage.
- The depth of inundation at an index location in the interior floodplain provides a measure of the consequences of closure failure. Floodplain inundation depth or stage can be related to economic damages using conventional methods.
- For a given configuration of openings through the levee, the floodplain stage corresponding to a given peak river stage can be estimated using hydrologic and hydraulic analysis techniques.
- The damage threshold stage, defined as the maximum floodplain stage at which damages are not significant, provides a lower limit for the probability curves.
- The stage of the river provides an approximate upper limit on the maximum possible interior stage.

The basic approach is, for a given river stage, to first define the two extreme points of the exceedance probability versus floodplain stage curve. Additional intermediate points are then developed as judged appropriate to approximate the shape of the curve. For a given river stage, the failure scenarios defining the extremes of the exceedance probability versus interior stage relationship are as follows.

- In the no damage (ND) scenario, all closures are performed such that the interior stage is less than a specified threshold required for damage to occur. Either the closure of all openings is completely successful or at least successful to the extent that any flows through the openings are too small to cause significant inundation.
- The worst case or complete failure (CF) scenario consists of flow through one or more openings inundating the otherwise protected floodplain to the maximum stage possible.

In some studies, the river stage may be used as an estimate of floodplain inundation depth for the worst possible closure failure. In other cases, hydrologic and hydraulic models may be used to determine a lesser floodplain stage for the worst possible closure failure. In some cases, for the downstream reach of a levee, the highest possible interior stage may exceed the river stage, reflecting flows through an opening located upstream ponding behind the levee further downstream.

Estimates of the conditional probabilities, $P(ND|S)$ and $P(CF|S)$, associated with the nondamaging (ND) and complete failure (CF) scenarios define the extremes of the probability curves for given river stages (S). A linear connecting of these two points provides a probability curve that might be adequate in some cases. However, since the frequency-stage relationship is logically nonlinear, additional points can be developed to better define a piece-wise linear approximation of the nonlinear relationship. For simplicity, the probability curves illustrated by Figure 5-7 are treated as linear segments defined by these points. Linear interpolation may also be used to determine probabilities for river stages between those developed.

One, two, or more partial failure scenarios are formulated and evaluated as necessary to define the shape of the probability curves. Partial failure scenarios result in specified levels of floodplain inundation between the two extremes. The conditional exceedance probabilities associated with the partial failure scenarios are expressed in terms of the probability that the interior floodplain inundation stage equals or exceeds a specified level, for a given river stage. The closure failure probabilities will be zero for river stages below the bottom of the lowest opening. Typically, for river stages above the top of levee, failure probabilities of 100% will be assigned to the worst case floodplain stage. This could be altered to reflect possible emergency raising of levees and closure structures with sandbags or other means during a flood event.

b. Closure Risk Estimation. Risk consists of both the likelihood and consequences of closure failure. The evaluation of consequences of failure is based on hydrologic, hydraulic, and economic investigations that may vary in level of detail and accuracy depending on the scope of the study. The hydrologic and hydraulic analyses required to determine the inundation conditions to result from postulated closure failures will typically be complex. However, the required modeling capabilities are available, including, if needed, steady or unsteady two-dimensional flow models. The floodplain stage will depend upon a variety of factors including the hydraulic characteristics of the openings, duration of flow through the openings, coincident flooding from interior runoff, topography and hydraulic characteristics of the interior floodplain, and the capabilities of the interior flood damage reduction system.

Estimating probabilities for closure failures has to be largely subjective. Little, if any, empirical data exists on past failures. There is also little or no past experience in estimating closure failure probabilities. Past practice has been to neglect this aspect of risk in the quantitative analysis, which is equivalent to assuming a zero closure failure probability. However, the probability estimates should be based on a thorough understanding of the circumstances and features of the particular project. Planning and analysis studies should be performed to the extent feasible to support the probability estimates. Closure risk analyses will rely largely upon the studies that are performed for other purposes anyway.

The river stage-frequency, river stage-discharge, and floodplain stage-damage relationships are already being developed, regardless of the closure analysis approach adopted. The following types of analyses may be performed specifically to provide a basis for evaluating closure risk. These types of studies support development of the stage-probability relationships illustrated by Figure 5-7 as well as otherwise developing an understanding of the closure risks associated with alternative plans. Similar analyses may be performed in evaluating the risk associated with disposing of interior flood waters addressed in Section 5-5.

- Formulation of alternative failure scenarios.
    - Identify factors contributing to possible failures.
    - Develop a set of failure scenarios, in terms of specific openings and combinations of openings experiencing various degrees of closure failure under various circumstances. (What conceivably could go wrong?)
    - Rank the relative likelihood of alternative failure scenarios. (Which closures or combinations of closures are most likely to fail under what circumstances?)
- Warning and response time analyses.
    - Hydrologic studies to evaluate warning times and associated reliabilities.
    - Design and operational planning studies to estimate response times and associated reliabilities.
    - Evaluation of reliability of flood monitoring, forecasting, and warning systems.
- Evaluation of reliability of facilities and equipment.
- Evaluation of institutional capabilities for project maintenance and operation.
- Hydrologic and hydraulic studies to delineate consequences of closure failures.
    - River hydrology and hydraulics.
    - Interior watershed hydrology and hydraulics.
    - Analysis of coincident river stage and flooding from interior runoff.
    - Routing of river flows through levee openings for postulated closure failures.
    - Interior flood inundation delineations for postulated closure failures.

c. Steps in Developing Conditional Failure Frequencies. Development of the frequency-stage relationships illustrated by Figure 5-7 for a particular plan involves the following tasks.

- Selection of one, two, or more river stages to represent the range from the bottom of the lowest opening to the top of levee.
- Determination of the conditions of floodplain inundation from interior watershed runoff to combine with each river stage.
- Definition of the extremes of floodplain stage.
  - Threshold damage stage based on stage-damage relationship.
  - Maximum stage either set at the river stage or determined based on hydraulic models.
- Estimation of probabilities associated with the extremes of the floodplain stages for each of the river stages. The probability estimates are based on the judgement of a planning team having a thorough knowledge of the river basin and flood protection project based on the types of studies noted above.
- Development of one, two, or more additional points on the curves to approximate the shape of the exceedance frequency versus floodplain stage relationships.
  - Hydrologic and hydraulic analyses of different types of closure failures and resulting floodplain inundation stages.
  - Selection of representation floodplain inundation stages.
  - Estimation of associated probabilities. These probability estimates again are based on the judgement of a planning team having a thorough knowledge of the project and flood characteristics of the river basin.
- Sensitivity analyses.

Sensitivity analyses should be performed to develop an understanding of the effects that the closure failure probability estimates have on the information developed and the decisions made in the planning and design process. Sensitivity analyses may be performed to test the effects of variations in probability estimates on various study results such as average annual damages, damage-frequency and closure failure-frequency relationships, and the selection of the recommended plan. The sensitivity to factors other than probability estimates may be evaluated as well.

5-8. Concluding Remarks

Incorporation of the risk of failure of closure structures and interior facilities in the risk-based analysis approach involves evaluation of both the consequences and probabilities of failure. The evaluation of consequences of failure is based on hydrologic, hydraulic, and economic investigations that may vary in level of detail and accuracy. Estimates of the probabilities associated with failures are necessarily subjective and very approximate. Little if any empirical data regarding failure frequencies are available. Probability estimates must be based on informed professional judgement, that should reflect a thorough understanding of the project conditions and circumstances derived from various analyses. Carefully considered risk estimates provide a better basis for decision-making than the conventional practice of assuming zero probability of closure failure in the quantitative analyses. Incorporation of closure risk in risk-based analyses, in combination with sensitivity analyses, can provide a significantly improved understanding of the alternative plans being investigated.

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