CECW-CE

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

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Manual
No. 1110-2-1413

24 August 2018

Engineering and Design
HYDROLOGIC ANALYSIS OF INTERIOR AREAS

1. Purpose. The purpose of this manual is to provide guidance in hydrologic analysis of interior areas for planning, design investigations, and flood risk reduction. The document was developed to supply the U.S. Army Corps of Engineers (USACE) field offices with procedural and technical guidance in performing hydrologic assessments of interior areas.

2. Applicability. This manual applies to all military or civilian locations where interior flood problems exist.

3. Distribution Statement. Approved for public release, distribution is unlimited.

4. References. Required and related publications are in Appendix A.

5. General. The procedures described herein provide information of interest to planners and designers of interior systems involving flood loss reduction measures and actions. Interior area investigations are differentiated from other studies only by the uniqueness of the hydrologic analysis requirements for the flood loss reduction measures commonly studied. Interior area planning studies are an essential aspect of feasibility studies. Although facilities and costs may at times be small components of a major line-of-protections project, the elements are often major items in the negotiated local sponsor agreements and can represent a significant proportion of local costs.

FOR THE COMMANDER:

2 Appendices

(See Table of Contents)

KIRK E. GIBBS
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Chief of Staff

*This Engineer Manual supersedes EM 1110-2-1413, dated 15 January 1987.

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CHAPTER 1

Introduction

1-1. Purpose. The purpose of this manual is to provide guidance in hydrologic analysis of interior areas for planning, design investigations, and flood risk reduction. The document was developed to supply the U.S. Army Corps of Engineers (USACE) field offices with procedural and technical guidance in performing hydrologic assessments of interior areas.

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CHAPTER 2

Overview of Interior Features

2-1. Overview. This chapter provides an overview of basic concepts and study considerations pertinent to hydrologic studies of interior areas.

2-2. Basic Concepts.

a. Interior Area. An interior area is defined as the area protected from direct riverine, lake, or tidal flooding by levees, floodwalls, seawalls, or low depressions or natural sinks. Figure 2-1 is a conceptual illustration of an interior area and Figure 2-2 shows sample attendant physical works.

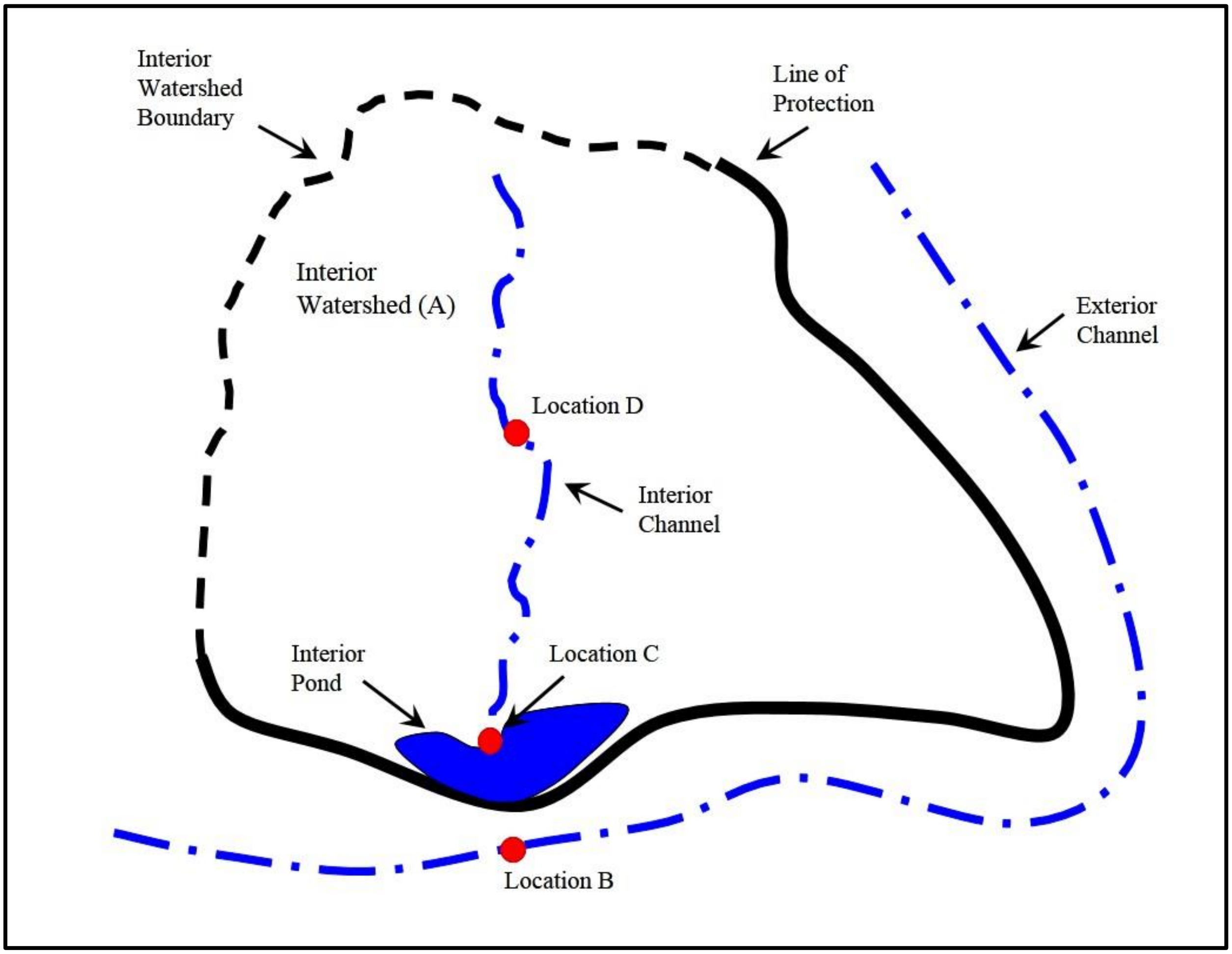

Figure 2-1. Interior area schematic.

b. Line-of-Protection. The line-of-protection is generally the levee or wall associated with an interior area. The line-of-protection excludes flood water originating from the exterior but normally does not directly alleviate flooding that may subsequently occur from interior runoff. In fact, the line-of-protection often aggravates the problem of interior flooding by blocking drainage outlets. Protected interior areas formerly flooded from the river, lake, or coastal area by slowly rising flood waters generated from regional storms, may now be subject to flooding from events that are more localized, occur more suddenly, and provide less warning. The flooding may be aggravated by coincident high-river, lake, or coastal stages. The interior flooding that results may be of the nuisance variety (shallow, temporary flooding), but can be in an extreme case as dangerous (or more so) as the situation without the levee.

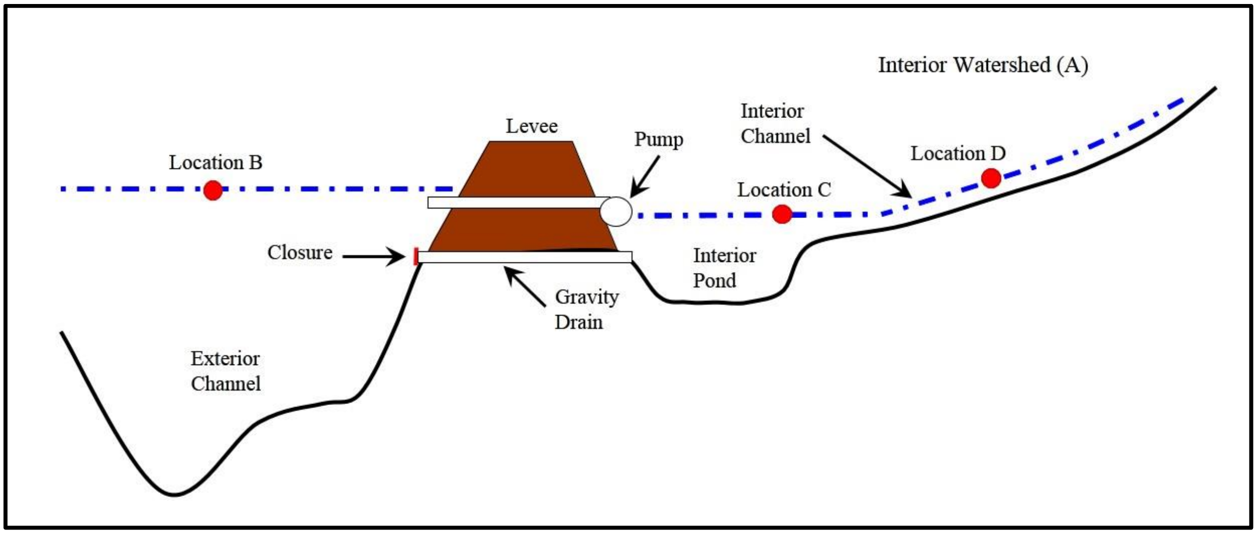

Figure 2-2. Profile view of simple interior system: state of system at Location C is influenced by interior runoff (Watershed A) and exterior stage at Location B.

c. Purpose of Interior Area Facilities. Interior flood waters are normally passed through the line-of-protection by gravity outlets when the interior water levels are higher than water levels of the exterior (gravity conditions). The flood waters are stored and/or diverted and pumped over or through the line-of-protection when exterior stages are higher than that of the interior (blocked gravity conditions). Gravity outlets, pumping stations, interior detention storage basins, diversions, and pressure conduits are primary measures used to reduce flood losses within interior areas. (See Section 2-3 for a description of each of these measures.) Other structural and nonstructural measures, such as reservoirs, channels, flood proofing, relocations, regulatory policies, and flood warning actions, may also be integral elements of interior flood loss reduction systems. Figure 2-3 is an aerial photo of actual line-of-protection and pump station.

d. Objective. Interior areas are studied to determine the specific nature of flooding and to formulate alternatives. Alternatives can evaluate many different criteria, including enhancement to the national economy, life safety, enhance the environment, social well-being, and regional development. The selected plan for implementation is the one that best meets these objectives and is discussed in ER 1105-2-100.

e. Studies. Interior area investigations are differentiated from other studies only by hydrologic analysis factors and the uniqueness of commonly implemented flood risk reduction measures. The study process and types of studies conducted to plan and design flood risk reduction actions are identical to those of other investigations. These studies include planning investigations, survey reports, and other forms of feasibility studies, design studies (Design Documentation Reports and Engineering Documentation Reports), and similar studies for small projects under continuing authorities. Analysis of interior areas is relevant to formulation and evaluation procedures, level of protection considerations, and hydrologic, economic, environmental, and social assessment criteria as established by present federal planning and design policies and regulations.

Figure 2-3. Aerial photo showing line-of-protection and associated interior drainage facility.

f. Importance. Interior area planning studies are an essential aspect of feasibility studies. Although facilities and costs may at times be small components of a major line-of-protection project, the elements are often major items in the negotiated local sponsor agreements and can represent a significant proportion of local costs.

2-3. Typical Interior Area Flood Risk Management Measures.

a. Gravity Outlets. Gravity outlets are effective means for draining an interior pond during the condition when the stage in the exterior channel is lower than the stage in the interior pond. When the case is reversed, in nearly all cases, the gravity outlet on the exterior side will have a flap gate closure (or lift gate closure). The flap gate will close when the water surface in the exterior river exceeds the stage in the interior pond. A gravity outlet is depicted in Figure 2-2.

b. Pumping Stations. Pumping stations are beneficial to drain interior ponds when there is no means to add another type of outlet and in cases when the exterior channel stage is greater than the interior pond stage. Typically, at an interior drainage facility both gravity drainage and pump stations are available. In the case when the flap gate would be closed on the exterior side the pump plant can evacuate the water in the interior pond. In conditions where the flap gate is open the pump plant can still be operated to help quickly evacuate the water in the interior pond. This may be the case during a very intense rainfall on the interior. A line-of-protection with a pump plant and gravity drain is depicted in Figure 2-2.

c. Interior Detention Storage Basins. Most interior pump stations will have a detention basin associated with it. The storage basin provides a location to store water as it runs off and can wait until it is pumped out. This may allow the pump operators to use smaller, more efficient pumps. The detention basin is also a way to store water to prevent or reduce local flooding when intense rainfall results in runoff that exceeds the capacity of the pump station. Figure 2-3 shows a combined channel and detention basin on the interior side of the pump.

d. Diversions. In some cases water can be diverted to another basin or to another location where additional drainage facilities can be used. Diversions can be designed to remove water from a detention basin before the water in a basin rises to a level that could cause damage. Diversions can also be used to remove water from an upstream collection point to another area that is not prone to flooding.

e. Pressure conduits. Pressure conduits include pipes and closed conduits that convey interior flood waters through the line-of-protection with internal pressure. Pressure conduits are a type of gravity outlet. These are generally buried fairly deep but still exit into the exterior stream. The pressure on the interior side can be generated by either head or from a pump. As with a gravity outlet, these will have some type of closure structure on the exterior side.

2-4. Complexity of the Interior Storm Water Management Problem. Hydrologic analysis of interior areas is complex because of interior flooding combined with uncertainty of stages on the exterior side of the line-of-protection. The investigation is often difficult. Records may be scant or nonexistent, land use (and thus runoff) may have changed from the past and is often continuing to change, natural drainage paths have been altered, and coincident flooding (a technically complex subject) is the common situation. Areas are generally small (less than ten square miles, though some are much larger) and the measures that should be considered are numerous. Chapter 4 discusses methods for analyzing interior areas.

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CHAPTER 3

Overview of Interior Area Hydrologic Studies

3-1. Overview. This chapter gives an overview of the approaches, strategies, and requirements of interior hydrologic engineering studies.

3-2. General Study Considerations.

a. Overview.

(1) Development of the hydrologic engineering study strategy, as captured in a Hydrologic Engineering Management Plan (EP 1110-2-9), is an important first step in producing quality technical results that are needed. Figure 3-1 is a schematic of steps that can assist in formulating the hydrologic study.

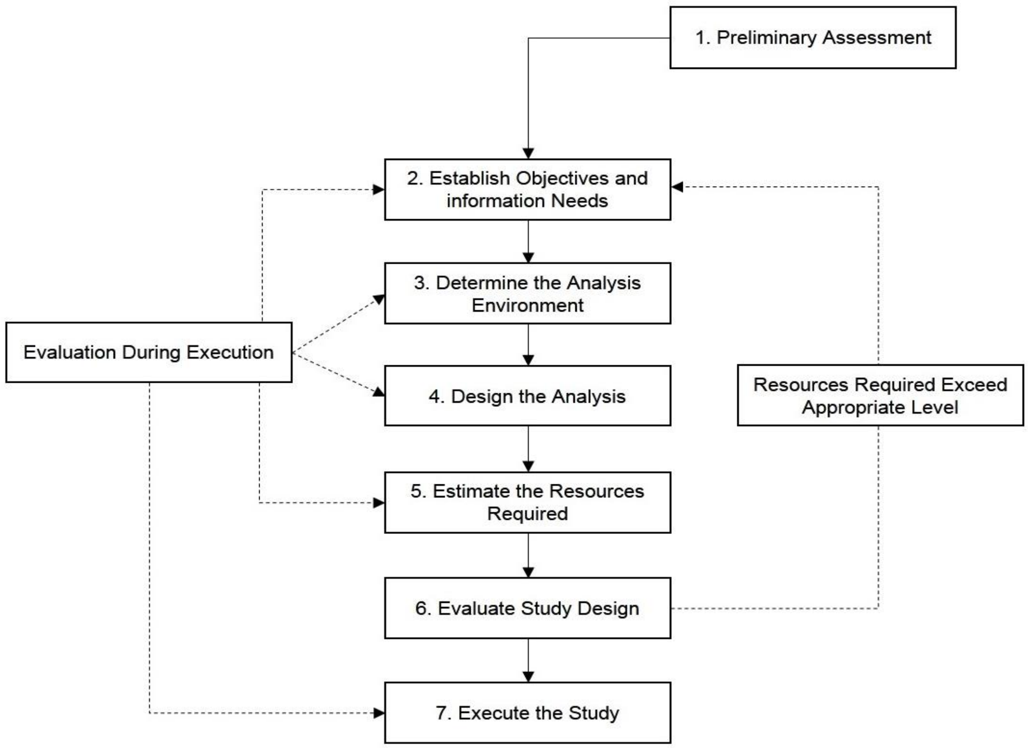
(2) Study resources include manpower, schedules, and funding allocations for the various participants in the study. Resource allocation should be a coordinated effort among the study manager and representatives of the various elements. Under some circumstances, adjustments in scope of the hydrologic aspects of the study to meet resource allocations may be accomplished by reducing the number of alternatives investigated or by the modification of the analysis procedures. Appropriate detail and scope must be maintained to meet required guidelines, regulations, and study procedures. Compromises between the study coordinator and the participant in resource allocations requirements may be required to meet these objectives.

b. USACE Approach to Flood Risk Management.

(1) The USACE approach to solving a flood risk management problem is a sequential process that involves planning, design, construction, and operation. Planning studies are performed according to the SMART (Specific Measurable Attainable Risk Informed Timely) Planning Principles. The level of study effort should align with these principles which promote balancing the level of uncertainty and risk within the level of detail of the study. The level of detail required to make planning decisions will grow over the course of the study, as the study team moves from an array of alternatives to a single recommended alternative. Details of SMART Planning Principles can be found at http://planning.usace.army.mil/toolbox/smart.cfm.

(2) A range of alternative plans will be identified at the beginning of the planning process and screened and refined in subsequent iterations throughout the planning process. The plan selected for design and implementation is the one that best meets the project's economic, environmental, and social objectives.

c. Information about Planning Process. Information about the planning process-formulation, evaluation, and selection of alternatives is provided in ER 1105-2-100. The



Steps

1. Perform Steps 1 through 6 in an abbreviated manner to formulate the relationship between overall interdisciplinary study objectives, strategy and the study resources required to meet these objectives.

2. Assist in formulating overall study objectives and strategy. Identify decisions required to carry out the strategy and meet the objective. Establish information required to support these decisions. Be specific as to what, where, when, quantity, and quality.

3. Assess the physical (meteorological, topographic, infrastructure, etc.) conditions influencing the study design. Determine the data that is available and its quality.

4. Develop an analysis strategy and select procedures that will provide the information required to meet the overall study objectives in a manner that effectively utilizes resources. Establish requirements for data to be acquired, sources, and quality.

5. Develop a detailed work plan to carry out the analysis designed in Step 4. Be specific as to who, what, where, when, how, quantity, and quality. Document the analysis design, working plan, and resource requirements. Tabulate resources needed to carry out each task and provide each information requirement.

6. Evaluate the analysis design, resources required, the study/design objectives, and make adjustments as considered appropriate. If the resources required (established in Step 5) exceeds what is considered appropriate then return to Step 2 and re-establish overall study objectives; do not change the analysis without adjusting the objectives.

7. During execution periodically evaluate the objectives, analysis, and resources; make adjustments as considered appropriate.

Figure 3-1. Formulation steps.

analysis of the nature and extent of real estate requirements is described in ER 405-1-12. Cost estimating is described in ER 1110-2-1302.

d. Level of Detail. The level of detail should be commensurate with the study purpose and other technical elements. The level of detail of the engineering efforts are described in ER 1110-2-1150 and should be in line with the SMART Planning Principles.

e. Scoping and Scaling Issues. In order to perform an interior analysis it is important for the analyst to understand the scale and scope of the problem. Input should be solicited from all members of the study team. This will help define what questions need to be answered and what geographic aspects define the extent of the study scope.

f. Risk Analysis Framework. The interior analysis and alternatives formulation must be executed under a risk analysis framework as presented in ER 1105-2-101 and EM 1110-2-1619 and as summarized in Chapter 7.

g. Systems Approach.

(1) When performing an interior analysis it is important to analyze the interior features as a system. Each of the pieces of the system operates in concert with each other and any analysis should take this into consideration. This system would include operating rules for a pump station and how those rules reflect stage in the interior pond and the relation with exterior stages.

(2) The analysis also should look at how the interior system could impact the exterior flows and stages. An increase in exterior stages could, in turn, impact resilience of the exterior system. This may not be important if the new interior features are being added due to the construction of a new line-of-protection structure. Generally, the new protection structure combined with new interior facilities, will, generally decrease the with-project interior contribution when compared to the without-project flows. However, this should be verified.

(3) In a case where new interior features are being added to an already constructed line-of-protection, the impact on exterior stages must be identified both upstream and downstream of the line-of-protection. Implementation of these measures must also meet criteria defined in Executive Order 11988 and other existing federal policy.

h. Analysis of Without-Project Conditions.

(1) Without-project conditions (current and future) for the study area consist of measures and conditions presently in place at the time of the study. Analyses are performed for with and without flood risk reduction measures in place (with-project and without-project conditions, respectively), the difference representing the impact of the project. Existing measures, implemented prior to the base year, and measures authorized and funded for construction completion prior to the base year are assumed to be in place and included for both with- and without-project conditions, as described in the ER 1105-2-100. If it can be clearly shown that

implementation of authorized measures is unlikely, then the measures should not be considered as in place for the without-project condition.

(2) Determination of existing without-project conditions is an important aspect of the study process. The without-project condition is the condition most likely to prevail in the absence of the plans under investigation by USACE. Existing flood risk reduction projects should be considered in place with careful consideration given to the actual remaining economic life of existing structures. Flood hazard plans authorized for implementation, but not yet constructed, should be considered in place unless it can be clearly shown that implementation of the measures is unlikely.

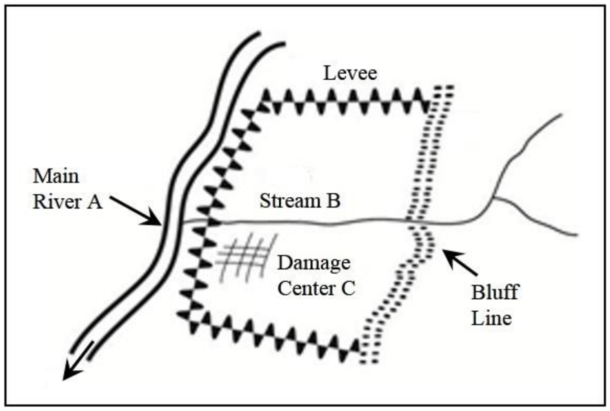
(3) Assessment of the existing without-project conditions must be of sufficient detail to establish viable economic (cost and flood damage), social, and environmental impact assessments of with-project conditions without further refinements throughout the remainder of the planning process.

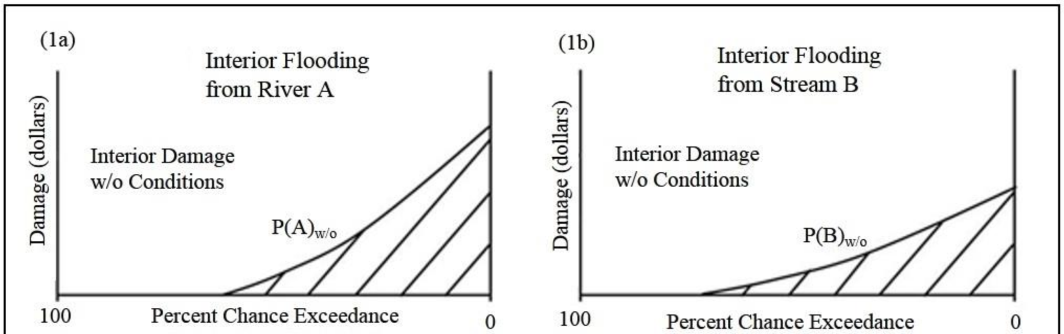
(4) Future condition analyses are performed for the most likely future development condition projected to occur without the project. The impacts of implementing the project (the with-project condition) are determined by comparisons to the without-project condition. Specified future time periods are assessed. Sensitivity analyses may also be desirable or required to determine the stability (viability and operation) of measures and plans for other possible alternative future development scenarios. The basis for projecting changes in the existing conditions must be clearly stated. Projects must be based on supportable information.

i. Analysis of Urban and Agricultural Areas. There is no distinction in the planning and design study processes between urban and agricultural areas. There is also no direct distinction between performance standards for urban and agricultural areas. However, urban areas often produce throughout the study process the need for higher levels of protection than agricultural areas, because the consequences of flooding are likely to be of greater social concern and solutions may introduce more significant environmental problems. As a result, studies of urban interior areas often surface a more complex mix of alternatives and measures based on economic, social, and environmental factors than agricultural areas, which typically yield systems that produce maximum net economic benefits. This does not preclude, however, the need throughout the study process for careful consideration of potential social and environmental impacts for agricultural areas.

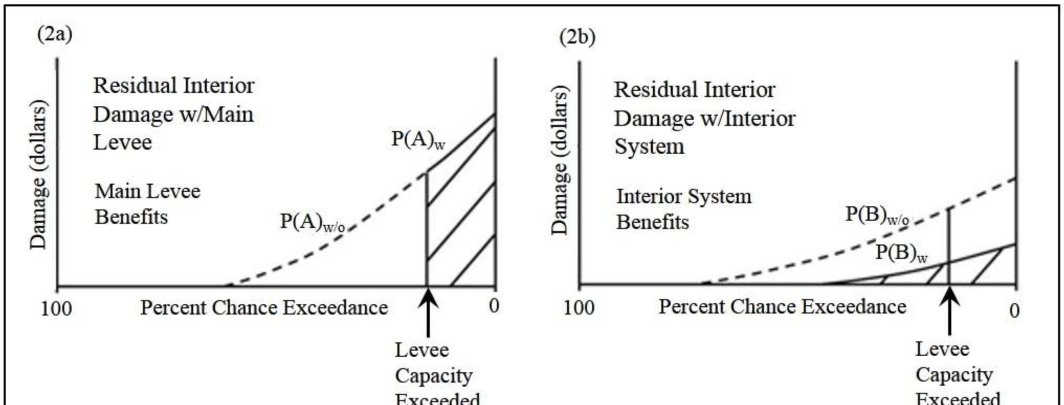
j. Flood Damage Evaluation Concepts.

(1) Flood damage evaluations of interior areas are complex. Figure 3-2 presents a simplified conception of the damage probability relationships assuming complete non-coincidence. The interior area is defined by the levee alignment and where that levee ties into high ground shown in the figure as the bluff line.





Condition 1: Without-project conditions – No risk reduction measures are in place. Total damage function for the potential damage center equals damage from Main River A [P(A)] plus damage from interior flooding along Stream B [P(B)] (since these events are non-coincident, the probability of intersection is zero).



Condition 2: Without-project conditions – Implementation of main levee (2a) and interior flood risk reduction measures (2b). Benefits of main levee are equal to area under curve $P(A)_{w/o}$ minus area under residual damage curve with levee in place. Benefits of interior system are equal to area under curve $P(B)_{w/o}$ minus the area under residual damage curve with interior system in place. Total benefits equal the sum of the two benefit values. Residual damage is similarly the sum of the residual values.

Figure 3-2. Flood damage-frequency relationship concepts.

(a) In Figure 3-2, Condition 1 displays the total damage frequency function for Damage Center C for the Without-Project Conditions. Without-Project Conditions is defined as without the main levee or floodwall and without any interior flood risk reduction measures in place. The damage-frequency relationship for Damage Center C is equal to the sum of the individual functions for the Main River A and interior runoff (Stream B). There is no intersection in damage functions between Main River A and interior runoff (Stream B) because of the assumption of non-coincidence.

(b) In Figure 3-2, Condition 2 illustrates the resulting damage-frequency relationship after the main levee or floodwall and interior flood risk reduction measures are implemented. The function generated in Condition 1a (Figure 3-2) for without-project conditions are truncated at the percent chance exceedance of when the main levee is no longer performing (indicated as "Levee Capacity Exceeded"). The levee capacity could be exceeded when the levee is overtopped or when a "design" elevation has been exceeded. Condition 2b (Figure 3-2) illustrates the damage-frequency function after implementation of proposed interior flood risk reduction measures, such as enlarged gravity outlets and/or pumping stations. No contribution to residual damage when the main levee is exceeded is included in Condition 2b. Assuming non-coincidence, the total damage reduction is the sum of the two. Residual damage is similarly the sum of the residual values.

(c) The reduction measures are treated separately when determining benefits. An assumption that benefits from the main levee, Condition 2a (Figure 3-2), are accrued up to the system capacity, (shown as Levee Capacity Exceeded. Figure 3-2) and once exceeded, damages occur as the without levee condition (hatched area) in Condition 2a (Figure 3-2). Levee capacity assumes there is no probability of failure for river stages below a threshold (this could be the top of levee or a levee design height where the levee completely fails when the design height is exceeded). The levee system benefits are represented by the area under $P(A)_{w/o}$ prior to the levee capacity exceeded level. The benefits for interior measures can be represented by the area between $P(B)_{w/o}$ and $P(B)_{w}$.

$$\text{Main Levee Benefits} = P(A)_{w/o} - P(A)_{w}$$

$$\text{Interior System Benefits} = P(B)_{w/o} - P(B)_{w}$$

(d) Incorporation of levee fragility information would change the shape of the residual damage-frequency function. Instead of a vertical line when the levee capacity is exceeded, the line would curve based on the fragility function defined for the levee. The damage-frequency curve for interior flooding from Main River A might look like Figure 3-3 when levee fragility is included (compare to Condition 2a, Figure 3-2).

(3) If complete coincidence had been analyzed, the benefits attributable to interior measures would be different. The benefits would be decreased by the hatched damaged frequency block in Condition 2a (Figure 3-2) that represents events exceeding the levee capacity level (Figure 3-4). This is because interior events more rare than the levee capacity level could not accrue interior

benefits because the levee would have already failed. In this case, the interior system benefits are decreased by the intersection of $P(A)_w$ and $P(B)_{w/o} - P(B)_w$.

$$\text{Interior Systems Benefits} = P(B)_{w/o} - P(B)_w - (P(A)_w \cap (P(B)_{w/o} - P(B)_w))$$

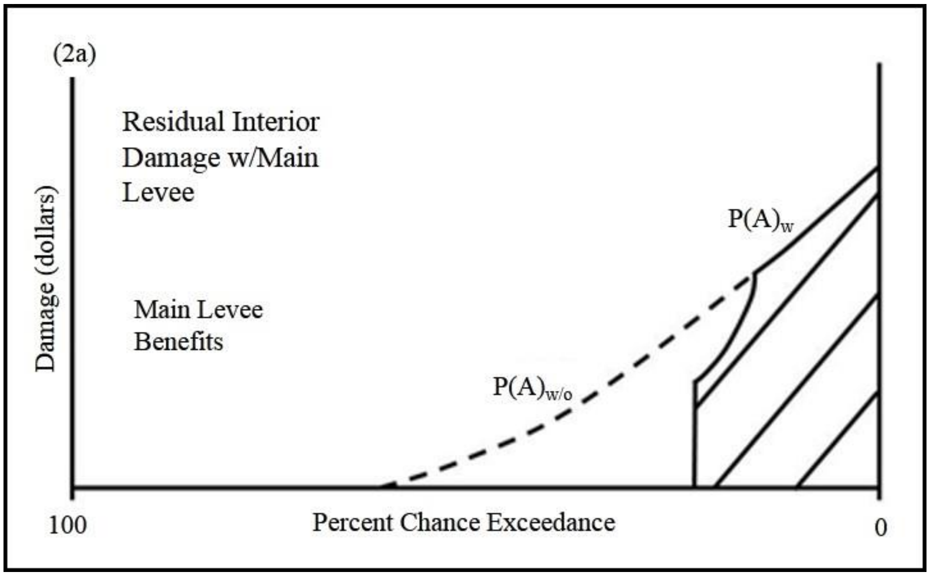


Figure 3-3. Inclusion of levee fragility.

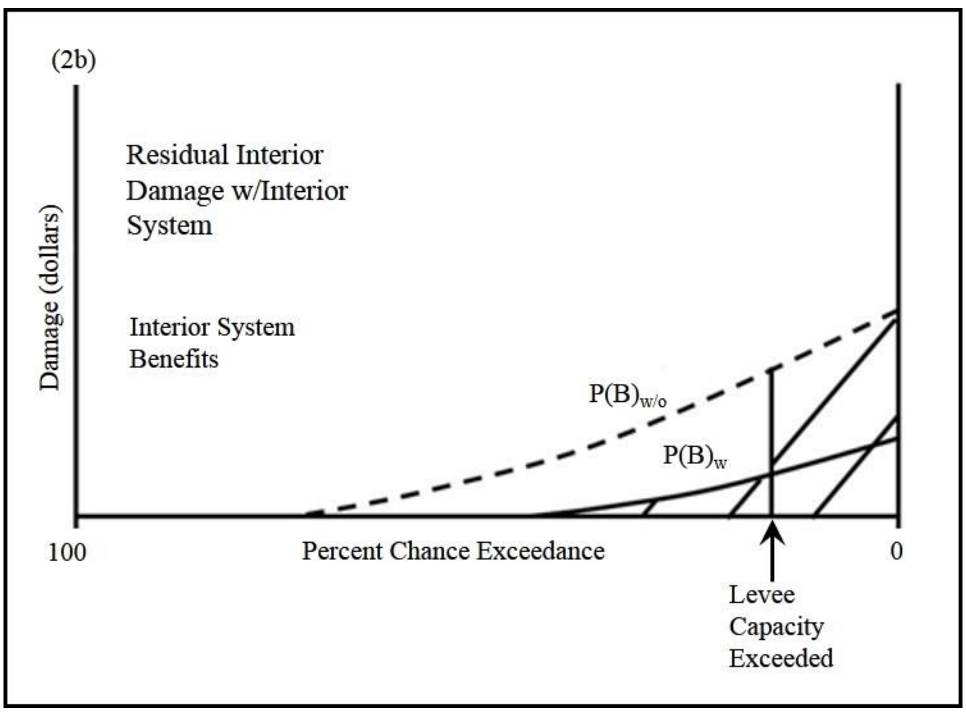


Figure 3-4. Interior system benefits when assuming complete coincidence.

(4) Economic analysis methodology is described in ER 1105-2-100 and ER 1105-2-101.

3-3. Hydrologic Engineering Requirements for Interior Area Planning Studies.

a. Purpose of Studies. Interior areas are studied to determine the specific nature of flooding and to formulate alternatives that enhance the national economy, while protecting the environment, social well-being, and cultural and historical values. Hydrologic analyses of interior areas must address the coincident nature of flooding at the line-of-protection for existing and future with-project and without-project conditions.

b. References for Requirements. Specific hydrologic engineering requirements for formulating and properly evaluating interior area plans are provided in EM 1110-2-1419. Selected concepts and requirements relevant to hydrologic studies of interior areas are described below.

c. Minimum Facilities.

(1) The hydrologic study strategy is formulated on the premise that interior facilities (that will be a component of the recommended plan) will be planned and evaluated separately (incrementally) from the line-of-protection project. The major project feature (levee/floodwall) is conceptually divided from the planned interior facilities by initially evaluating a "minimum" interior facility considered integral to the line-of-protection. If a levee/floodwall is in existence, the minimum interior facility is presently in place, and no special efforts are required to establish the separation. If a levee is being proposed (planned), the minimum facility must be formulated and the evaluation of the line-of-protection benefits performed with the facility in place. The residual interior flooding problem is the target of the interior facility planning efforts, and benefits attributable to the increased interior facilities will be the reduction in the residual damage.

(2) The minimum facilities are intended to be the starting point from which additional interior facilities planning will begin. The criteria suggested below for determining the minimum facility are intended to yield facilities that can be quickly and easily determined. The facilities will, with rare exception, be found inadequate upon further interior facility planning; thus, increased facilities will be formulated, evaluated, and included as a component of the recommended line-of-protection plan that is, in turn, an incrementally justified component of the overall flood control project. It is expected that the interior facilities included in the final plan will provide interior area flood relief for residual flooding.

(3) The minimum facility should provide interior flood relief such that during low exterior stages (gravity conditions) the local storm drainage system functions essentially as it did without a levee in place for floods up to that of the storm sewer design. If a local storm drainage system is in existence, then the minimum facility should pass the local system design event with essentially no increase in interior flooding. If no local system presently exists, but future plans

include a storm drainage system, it is reasonable to proceed as if it exists and its design capacity is consistent with local design practices.

(4) Minimum interior facilities will most often consist of natural detention storage and gravity outlets sized to meet the local drainage system. However, they may include other features, such as collector drains, excavated detention storage, and pumping plants, if these measures are more cost effective.

(5) Special situations may arise in which the minimum interior facility concept is simply not applicable. Examples may include coastal areas where a significant portion of the interior water comes from wave splash over the line-of-protection; alternatives for interior flooding that substantially reduce the volume of water arriving at the line-of-protection, such as diversions or line-of-protection realignment; and line-of-protection projects in which the interior facility is a significant element in the overall project or where the interior measures are integral to the project in such a manner that separation is impractical. In these situations, the analyst is encouraged to adhere to the concept of separable evaluation and justification as much as practically possible to ensure careful analysis of interior solutions. Where completely impractical, the reason should be documented and the analysis should proceed in a logical, systematic manner considering the line-of-protection works and interior facilities as a unit.

d. Existing Without-Project Condition System Layout. Specific criteria and considerations in laying out the study area are as follows:

(1) The system is assumed to be in place and operating as planned, if the line-of-protection (levee, floodwall, and seawall) is presently in place or authorized for construction.

(2) If the line-of-protection is not presently in place, its feasibility and specification will be determined based on appropriate formulation and evaluation procedures. The feasibility study will include plans of alignment of the line-of-protection which minimize the contributing runoff area to the interior. This requires special attention to tie back levees, diversions, and use of pressure conduits (EM 1110-2-1913).

(3) If the line-of-protection is not in place, a minimum facility will be formulated and considered as part of the line-of-protection system.

e. Existing Without-Project Condition Assessments. Hydrologic analyses of existing without-project conditions will be performed to develop the basis for which the interior facilities will be planned. The analyses provide flood hazard information (frequency, magnitude, elevations, and velocities) which are integrated into assessments of other study elements (e.g., flood damage, cost, social factors, and environmental factors). Hydrologic analyses include the development of information for estimating elevation-frequency functions (discharge or storage-based) at desired locations throughout the system. The general hydrologic strategy for analyzing existing without-project conditions is as follows:

(1) Assess available information.

(2) Perform field reconnaissance of the area: conduct interviews, survey data needs, gather historic event information, and determine physical and operational characteristics of existing components.

(3) Assess analytical criteria for performing the study, i.e., layout for line-of-protection and existing condition components, and determine subbasin and damage reach delineation and existing land use patterns.

(4) Analyze exterior stage conditions at existing or potential outlets of interior facilities.

(5) Develop rainfall-runoff analysis parameters for the interior areas as appropriate. Parameters include data required for computing basin average rainfall, loss rates, runoff transforms (empirical unit hydrograph methods, conceptual kinematic wave transform methods, and physically based overland flow routing methods), and channel routing criteria.

(6) Formulate and evaluate the minimum interior facility.

(7) Generate hydrographs for the interior system by rainfall-runoff analyses, combine flows, and perform channel and storage routings as required throughout the system. The coincident flood routings (interior and exterior stage considerations) through the line-of-protection at existing gravity or pressure outlet and pumping station locations may be performed separately or in conjunction with the other system analysis. Seepage contributions should be included if pertinent.

(8) Develop elevation (discharge or storage-based) frequency functions or event parameters (historic record analysis) at selected damage reaches and other locations. Wave overtopping of the line-of-protection should be included if pertinent.

f. Future Condition Assessments. Future without-project analyses repeat the hydrologic strategy and procedures defined under the existing without-project condition for the most likely future conditions. This includes both land use and conveyance system changes. Other future alternative land use conditions may be assessed if desired or necessary. Future land use development patterns and other actions may affect hydrologic loss rates, runoff transforms, and possibly natural storage and conveyance areas. These effects, including assumptions of encroachment, sediment, and maintenance requirements to maintain the functional integrity of the proposed project, must be determined and documented. Analyses of future with- and without-project conditions are normally developed and presented at decade intervals through the life of the proposed project.

g. Formulation and Evaluation. Hydrologic analyses of flood risk management measures are performed for several combinations of measures (plans), operation plans, and performance targets. The initial evaluation should assess the potential for improved operation of the existing system. If improved operation procedures are found to be attractive for the present system they should be detailed and incorporated as part of the existing system. The typical sequence of the feasibility analysis is to 1) evaluate increased gravity outlet capacity, 2) ponding, 3) pumping stations, 4) interceptor systems, and then other measures. Formulation and evaluation must be conducted under a risk analysis framework as discussed in Chapter 7.

h. Other Considerations for Interior Area Hydrology Planning Studies. Several important sub-problems must be resolved by the hydrologic engineer in the formulation and evaluation of proposed interior systems, such as exterior elevations for gravity outlet gate closure and pump on and off elevations. If sub-problems can be determined by independent analysis involving only hydrologic factors and the results do not significantly affect plans that are formulated and evaluated, then the hydrologic engineer should solve them. If the sub-problems interact in important ways with the measures being formulated, these technical sub-problems should be incorporated into the planning process that considers costs, benefits, and impacts of measures. Examining the sensitivity of the performance of planned interior facilities to variations in such factors is often useful.

(1) A basic concept is that the recommended plan will emerge from the planning process considering the full range of concerns and planning objectives. Costs and benefits will dominate, but other social, environmental, life safety and functional performance issues are important.

(2) The performance of the interior facilities over the full range of anticipated interior events, including those that exceed the system capacity, are particularly important. What happens when the system capacity is exceeded? Do excess waters rise slowly or rapidly? What is the warning time for evacuation? Can interior area occupants get into and out of the area as needed? What are the provisions for emergency services (police, fire protection, and medical service) and other life support requirements (food, water, shelter, and power)? Will the formulated facilities continue to function as planned under conditions that may prevail during the occurrence of a full range of possible interior storm events? The hydrologic engineer should participate in the decision process in these and similar items for which technical expertise is particularly helpful.

3-4. Hydrologic Engineering Requirements for Interior Area Design Studies.

a. Reference for Requirements. Requirements for preparation and processing of design reports are provided in ER 1110-2-1150.

b. Study Objectives. One design study objective is to provide refinement detail sufficient to meet construction and subsequent operation and maintenance criteria. Another objective is to perform cost effective assessments of the refinements and components while maintaining the integrity of the recommended plan. Hydrologic design analyses should interface with other design elements to achieve those objectives. This should include hydraulic design elements of the recommended plan such as the size, invert elevations, and development of the rating curves for gravity outlets, pumping station sump dimensions, and water surface profiles and flow velocities associated with proposed runoff conveyance system.

c. Selected Issues. Selected hydrologic considerations are described below. The items vary with each study.

(1) Pump station requirements, including pump start and stop elevations, selection of desired pump floor elevation and determination of the need for flood proofing above that floor elevation, and the extent of automation of the pump station operations to be commensurate with the extent of advance warning time. When considering the number of pumping units within a pump station, a minimum of two or preferably three pumps should be used such that sufficient station capacity is available when one pump fails. Refer to EM 1110-2-3102 for more information about pump station design.

(2) River data and criteria commensurate with gravity outlet capabilities, including selection of final gravity outlet gate closure elevations and the need for a manual or automated system of opening gravity outlets when interior pond stages exceed river stages.

(3) Detention storage requirements, including storage allocation for sediment, final interior stage-probability curves, duration, and depth data to determine potential hazards associated with ponding, and the real estate requirements (permanent right-of-way and/or flowage easements).

(4) Other hydrologic evaluations, including final assessment of impacts from interior runoff events that produce interior stages exceeding selected pond right-of-way, pump station floor elevations, and other existing development elevations, including the impacts from the standard project storm, and the determination of cofferdam levels for the construction of the interior flood control features (may include the development of seasonal stage-probability curves for anticipated construction schedules). Seepage can be a major consideration where external river stages remain high for prolonged periods.

(5) The action required to operate and maintain the proposed system, described in detail, including flood warning-emergency preparedness components and actions. The operations and maintenance requirements should be described by flood stage or elevation.

3-5. Reporting Requirements for Interior Area Hydrology Studies.

a. General Requirements. General reporting requirements for the several types of studies are described in ER 1105-2-100, and specific requirements for planning and design documentation are described in ER 1110-2-1150. ER 1105-2-100 notes, "Planning decision documents should be prepared in a timely and cost-effective manner, consistent with the size and complexity of the project. Likewise, the time and effort spent in technical and policy review and in responses to review comments should reflect the size and complexity of the project. Wherever possible, technical and policy review should be incorporated positively and proactively in early phases of the planning and documentation processes and throughout these processes, rather than at the end".

b. Requirements of Hydrologic Studies. ER 1110-2-1150 states that hydrologic studies facilitate the evaluation of economic and environmental impacts of alternatives: "These studies are required to determine the functional design and requirements of water resource projects and to establish channel capacities, structure configurations, levels-of-protection, interior flood-control requirements, the without-project conditions, and the project economic analyses. For flood reduction projects, it is equally important to address internal flood control requirements and residual flooding when evaluating alternatives. Physical and numerical modeling may be required in the feasibility phase to demonstrate that the proposed alternative(s) can be designed to satisfy project objectives and to determine project costs within the required level of accuracy".

c. Engineering Appendix to Feasibility Report. ER 1110-2-1150 describes the hydrologic reporting requirements for feasibility studies. The guidance states that such reports are to present the basis and results of hydrologic and hydraulic studies required for determining the functional design requirements of all water resource projects, and to explain the methods used, why the methods were selected, and the basic assumptions on which these studies are based. The engineering appendix should also provide basic data as appropriate and discuss the limitations of the collected data; to present results and conclusions; and, explain the data applies this information to design and real estate requirements. Appendix C of ER 1110-2-1150 provides a list of specific items that may be required, depending upon the type of project under development.

d. Design Documentation. Appendix D of ER 1110-2-1150 describes the requirements of the design documentation. Results of investigations, analyses, and calculations made for the design are to be included in this documentation. Such information may include refinements to project hydrology for specific features, determination of pertinent hydraulic design features, flow characteristics, discharge capacities, design water surface profiles, discharge coefficients and curves, other plotted data or tabulations, and results of hydraulic model tests.

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CHAPTER 4

Analysis Methods and Procedures for Riverine Interior Areas

4-1. General. Riverine interior areas are floodplains that are protected by a line-of-protection from overflow from an exterior channel. The line-of-protection obstructs discharge from the exterior channel. The line-of-protection can include a levee, floodwall, or both. A riverine interior area is illustrated in Figure 4-1. In Figure 4-1, the thicker solid line (black) represents a levee. The dashed line (black) is the upper boundary of the interior area watershed; the line-of-protection forms the lower boundary. Two channels are shown with dash-dotted lines (blue): the exterior channel from which the levee provides overflow protection, and an interior channel which "drains" the interior watershed. Under natural conditions (before construction of the line-of-protection), that interior channel conveys water from the interior watershed to the exterior channel. But, as shown, the line-of-protection blocks the discharge in the interior channel,

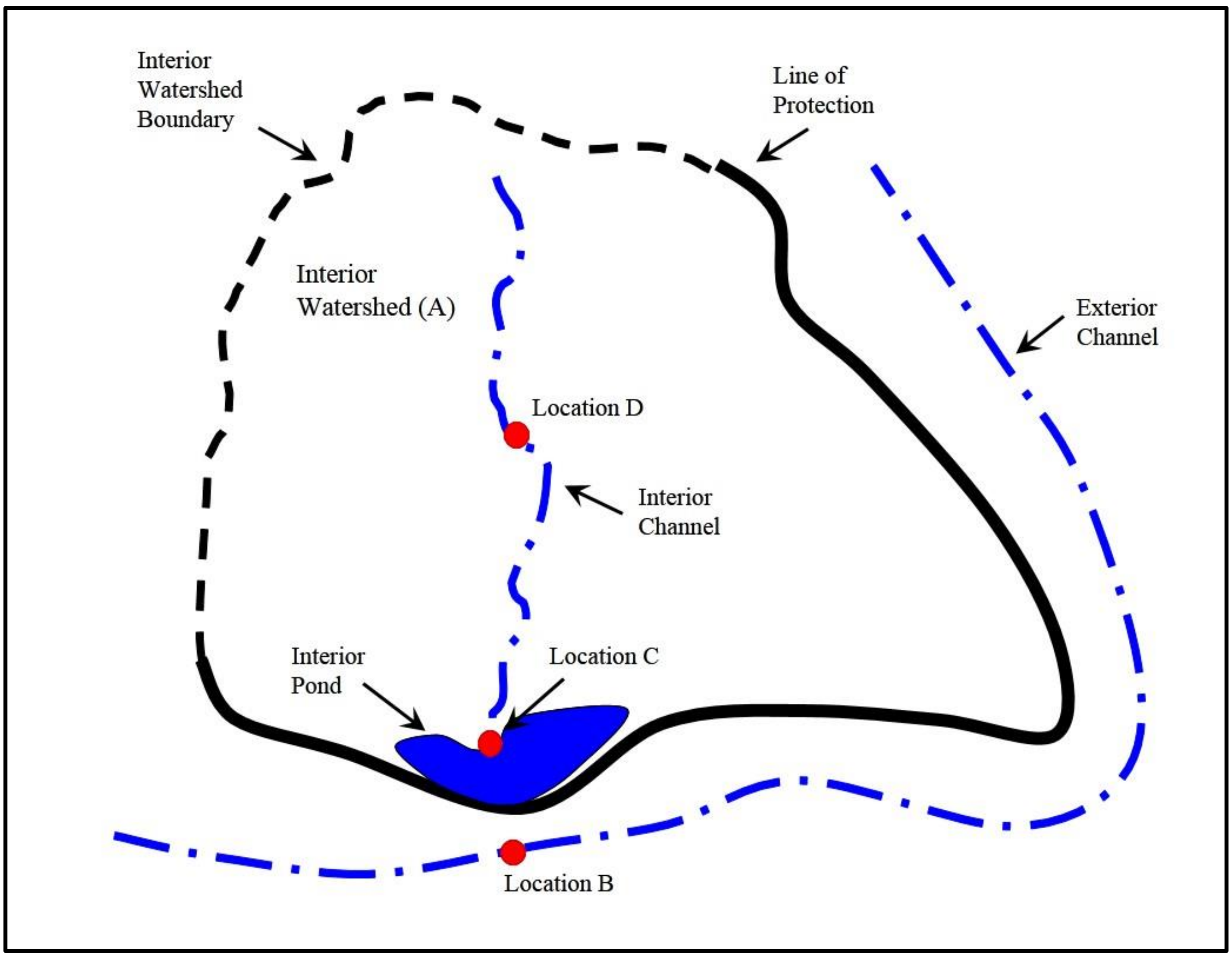

Figure 4-1. Plan view of interior system: line-of-protection prevents flooding from exterior channel but also obstructs natural drainage of interior watershed.

causing the water that would flow into the exterior channel in the absence of the line-of-protection to pond on the interior side of the levee. If that water is to drain to the exterior stream, the water must be managed and conveyed to the exterior channel through a gravity outlet or with pumping. Otherwise, the accumulated interior water will overflow interior channels or ponds and inundate the interior floodplain, causing property damage or injury.

a. Interior Analyses. Hydrologic analyses of interior areas are required for many types of USACE studies, including:

(1) Computing Expected Annual Damage (EAD) and other measures of risk for planning, designing, and operating features of the line-of-protection, including minimum facilities that are included as a part of the line-of-protection.

(2) Computing EAD and other measures of risk for planning, designing, and operating additional interior drainage facilities.

(3) Computing EAD and other measures of risk for describing the residual risk for the interior area.

(4) Mapping extent of inundation in the interior floodplain for various conditions.

(5) Computing life loss estimates for assessing public safety risk and for contributing to an emergency preparedness plan.

b. Sources. Sources of interior water include, but are not limited to the following:

(1) Runoff from precipitation on the interior watershed.

(2) Agricultural irrigation discharge and field runoff.

(3) Treated discharge from wastewater treatment facilities.

(4) Interior reservoir releases.

(5) Baseflow.

(6) Seepage under or through the line-of-protection that accumulates in the interior area

.

(7) Interbasin transfer of water that is diverted or overflows from an adjacent interior area.

(8) Overtopping or outflanking of the line-of-protection.

c. Interior Basin. A profile view of an interior basin with simple facilities for managing the water accumulated in the interior floodplain is shown in Figure 4-2. In this system:

(1) The interior channel terminates at the pond.

(2) Water accumulates in the interior pond until it can be conveyed to the exterior channel.

(3) The interior drainage facilities consist of an interior pond, a gravity drain (a closed conduit through which water flows without pumping) through the levee, and a pump.

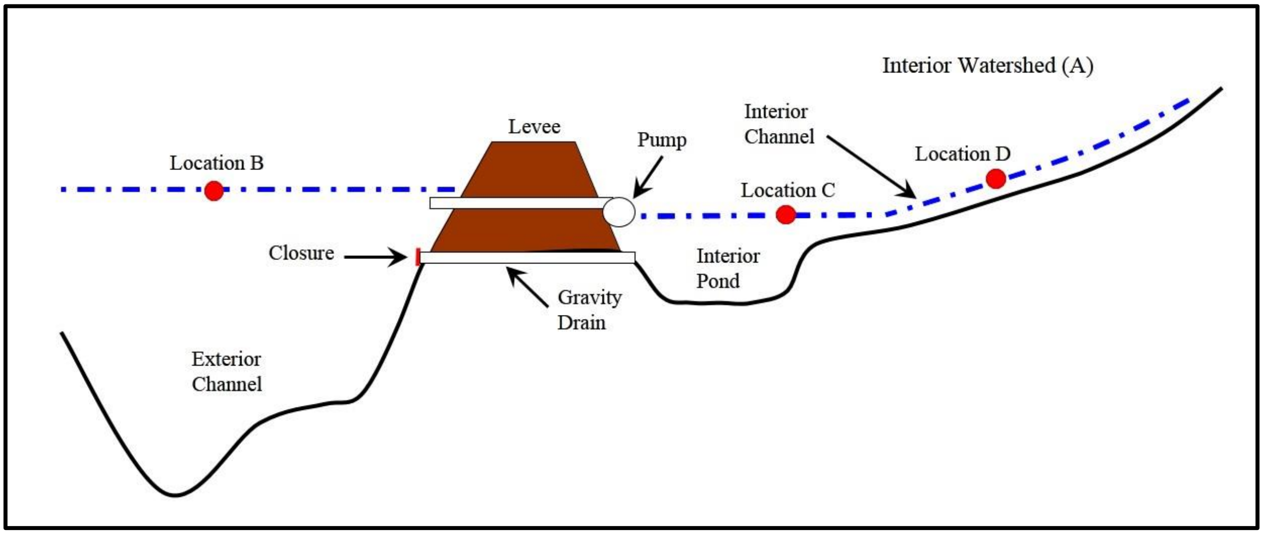

Figure 4-2. Profile view of simple interior system: state of system at Location C is influenced by interior runoff (Watershed A) and exterior stage at Location B.

(4) The drain (Figure 4-2) has a flap gate on the exterior (exit) end of the conduit that is normally closed. That closure prevents backflow of water from the exterior channel into the pond and onto the surrounding floodplain if the pond fills. Water from the interior pushes the flap gate open as it exits the conduit and flows to the exterior under appropriate conditions.

d. Exterior Stage Influence. The influence of exterior stage on interior stage is illustrated by three scenarios:

(1) When the gate is closed as illustrated in Figure 4-2, no water flows through the conduit, so storage in the pond remains the same or increases if inflow into the pond continues. The rate of pond storage increase depends on the inflow (the rate of runoff from the interior area). Thus the stage at Location C is influenced by the discharge in the (interior) channel and the stage in the exterior channel at Location B. With a high stage at Location B, the gate is closed, and the pond stage at Location C increases if discharge in the interior watershed (A) continues.

(2) If the stage at Location B is less than the stage at Location C, no water from the exterior channel will flow back through the conduit. In that case, the gate is opened and water will flow out of the pond into the exterior channel. The outflow depends upon the head differential (difference in pond and exterior channel stage), properties of the conduit, pond storage

characteristics, and the inflow to the pond. The stage at Location C, thus, is a function of inflow to and outflow from the pond, the latter of which is related to the exterior stage.

(3) If stage in the exterior channel at Location B is far below the interior pond stage at Location C, water will flow from the pond into the exterior channel, limited only by the hydraulic capacity of the conduit. In that case, storage and stage in the pond will be influenced most by the inflow, rather than by the exterior condition.

e. Combinations. Considering the scenarios described above, various combinations of events could yield the same pond stage at Location C. For example, if the exterior stage is high, resulting in gate closure and the inflow volume is small; the pond eventually may fill beyond capacity and overflow. Similarly, if the exterior stage is low, with gates opened, and the inflow volume is high, the pond may also fill and overflow. Both cases incur damage.

f. Non-Impacted Areas. Location D in Figure 4-2, is far upstream in the interior channel, out of the backwater of the pond. Stage at Location D is a function of the discharge in the channel only. Therefore, stage at that location can be estimated with a rating curve developed from observations or a model of channel hydraulics, without reference to the condition in the pond or exterior channel. In other words, stage at Location D is independent hydraulically of the state of the pond, and thus independent of the state of the exterior channel. Procedures for analysis of this simpler case are described in other guidance, including EM 1110-2-1415, EM 1110-2-1416, EM 1110-2-1417, and EM 1110-2-1419. This case is not considered further in this chapter.

g. Summary. The analysis of interior conditions, which depends on both interior discharges and exterior stages, such as conditions at Location C in Figure 4-1 is addressed. Specifically, Figure 4-1 describes and illustrates alternative methods for determining the stage-probability curve at a selected point in the interior area, considering the hydraulic interconnection of the interior watershed and the exterior stream. The analysis requires use of models and methods of watershed hydrology, statistical hydrology, open-channel hydraulics, and closed-conduit hydraulics. Relevant specialized terms are defined in this section in the context of interior analysis. For broader, more general definitions of these terms, the reader should consult EM 1110-2-1415 and other manuals and documents that focus on probability and statistics.

4-2. <u>Basic Concepts</u>. The occurrence of fluctuating water levels both exterior and interior to the line-of-protection is the aspect that makes interior area analysis unique. Several terms are used to communicate information about the nature of these occurrences and the occurrences represent important basic concepts. If the exterior and interior occurrences are such that a consistent relationship exists one to the other (to some degree, one can be predicted from the other), the interior and exterior events are said to be correlated. If the physical and meteorologic processes of the interior and exterior events are related to one another, the occurrences are said to be dependent. If the situation occurs that the interior and exterior events produce stages that coincide, e.g., the exterior is high when an interior event occurs, then coincidence is said to occur. These terms are discussed in more detail in paragraph 4-2a.

a. Inspection of the historic record is fundamental to determining important factors of correlation, independence, and coincidence. Establishing bounds on the consequences of decisions regarding these factors is an important analytic approach. Analyzing the two extremes of assuming complete and non-existent coincidence is generally helpful. Also, by determining the relative consequences of the assumption of independence, judgments regarding its importance to the study can be made. Within the framework of this information, the approach that will yield supportable conclusions will become more evident. Table 4-1 summarizes hydrologic analysis considerations for various levels of coincidence and dependency of interior and exterior conditions.

Table 4-1. Assessment of coincidence.

Coincidence	Dependence	Example/Comments	Analysis Considerations
(High)	(High)	Hurricanes, large regional events, interior and exterior areas of similar magnitude.	Blocked gravity outlet conditions are common. Conventional hypothetical frequency analyses often appropriate for urban areas.
	(Low)	Storm season of small interior area season coincides with snowmelt runoff of large basins.	Continuous record analysis methods or probabilistic approaches generally required. Gravity outlet is often blocked during critical interior events.
	(High) – (Low)	This range of coincidence is most common. Relatively high likelihood of interior and exterior events occurring simultaneously.	Continuous record analysis or probabilistic methods generally required. Gravity outlets may be blocked during critical interior events.
	(High)	Timing of interior and exterior events is such that the events rarely coincide. May be affected by operation of upstream project.	Considerable study may be required to identify this condition and to assume its existence in the physical process. Coincident hydrology generally appropriate.
(Low)	(Low)	Rare condition. Interior flooding rarely if ever coincides with high exterior stages. Studies generally limited to gravity outlet assessments.	Coincident interior analysis is not necessary.

b. Coincidence. Coincidence refers to the simultaneous occurrence of a specified interior condition and a specified exterior condition in a system. At one extreme it is possible, though not likely, that there will be complete non-coincidence, i.e., the two occurrences will never coincide and thus interior and exterior water levels will never be high or low at the same time. The interior analysis could be performed without consideration of exterior conditions, thus greatly simplifying the problem. The occurrences could be correlated and dependent/or independent, but it would not be important to the analysis approach. The following discussion and examples are limited to two variables, interior condition and exterior condition. Inclusion of additional variables, like flood season, storm type, storm duration and outlet blockage, are not included at this time as more research and modeling capabilities are needed to include these additional uncertainties in the analysis.

c. At the other extreme, it is possible, and somewhat more likely, that there will be complete coincidence, e.g., the two occurrences will always coincide so that high exterior levels are always present in the case of the occurrence of an interior event. The interior analysis can proceed without exterior analysis (by assuming blocked gravity outlets), since the conditions that exist for interior events are completely known. The occurrences would likely be correlated, although not necessarily dependent, but it would not be important to the analysis approach. Figure 4-3 illustrates the concept of coincidence. Figure 4-3(a) is a discharge hydrograph of the interior runoff, (Q_A), runoff from the interior watershed (Location A) in Figures 4-1 and 4-2. Figure 4-3(b) is an exterior stage hydrograph at Location B (Figures 4-1 and 4-2). For the time labeled t_1, the interior runoff and exterior stage that occur simultaneously are "connected" with a vertical dashed line. The interior pond stage would be a result of interior runoff and the simultaneous exterior stage, (Z_B). At time t_2, the simultaneous interior and exterior conditions are connected with a vertical dashed line, and again interior pond stage will be a result of the connected values. Interior runoff at t_1 is greatest; however, exterior stage is lower at t_1 than at t_2. Therefore, the greatest pond stage likely is a result of conditions at t_2, as in that case, discharge from the interior area through the line-of-protection will be restricted due to higher exterior stage. At t_3, the peak exterior stage occurs, but it is not coincident with a large interior discharge. Lower interior pond stages result from an occurrence of discharge at t_1 and stage at t_3 because these conditions are not coincident as they are at t_2. Coincidence can exist whether or not the interior and exterior occurrences are correlated or dependent.

d. Correlation. Correlation refers to the degree to which coincident interior and exterior events tend to be consistent in magnitude in a predictable manner. The concept is illustrated by Figure 4-4, displaying plots of interior runoff and coincident exterior stage at three locations. In each case, the annual maximum interior instantaneous discharge and the coincident instantaneous exterior stage are plotted. Each point represents the value for a single year, selected from the record of observations at these locations.

(1) Figure 4-4(a) illustrates a case of high correlation, and in this case that correlation is positive. This means that greater interior runoff is always coincident with a higher exterior stage. Note, however, that high positive correlation between interior runoff and exterior stage does not mean that high interior flow causes high exterior stage, or vice versa, or that both high interior flow and high exterior stage are caused by the same phenomenon. Analysis of the case illustrated in Figure 4-4(a) is straightforward. The exterior stage above which gravity discharge through the line-of-protection is no longer possible and is determined with hydraulic modeling, and the corresponding interior runoff is found by consulting the plot. Then for all lesser interior discharges, the interior stage is computed with gravity flow. For all greater interior discharges, interior stage is computed without gravity flow. Similar high, but negative, correlation is possible. In this case, high interior runoff would coincide with low channel stage. Analysis of this case would be straightforward, as well. Again, the threshold exterior stage for gravity flow

can be determined, and the corresponding interior runoff peak found. All events with greater interior peak discharge are analyzed with gravity flow and all events with smaller interior peak discharge are analyzed without gravity flow.

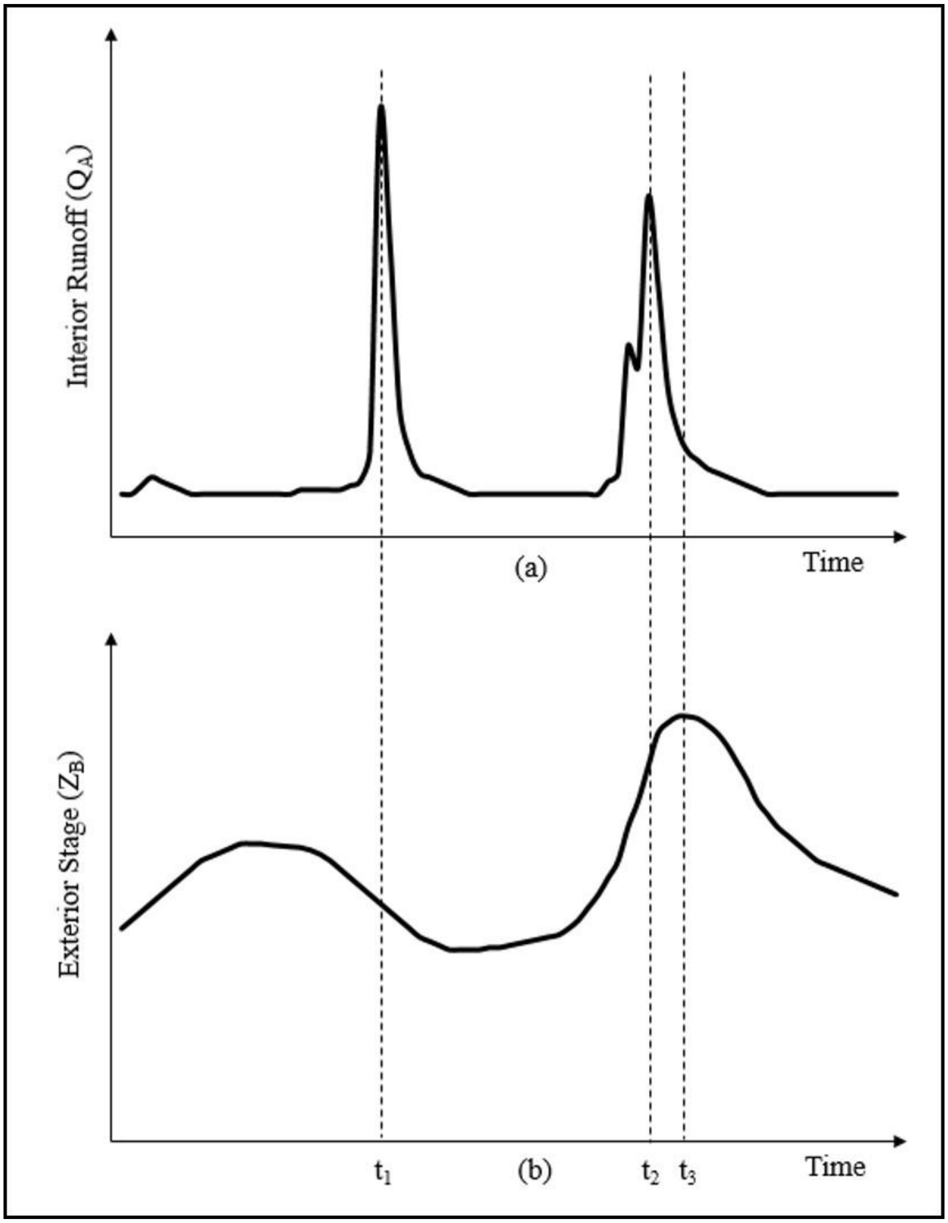

Figure 4-3. Interior and exterior hydrographs: discharge and simultaneous stage identified with dashed lines.

(2) Figure 4-4(b) illustrates a case in which no relationship of interior runoff and exterior stage is identifiable: high interior runoff is coincident with both high and low exterior stages, and no pattern is discernible.

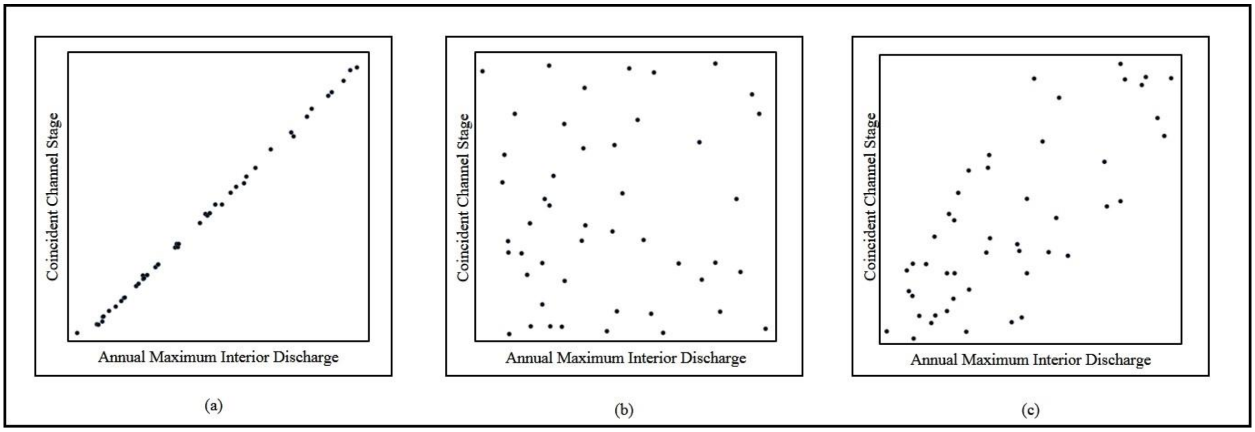

Figure 4-4. Illustrations of correlation of coincident interior and exterior conditions: (a) illustrates high positive correlation; (b) illustrates low or no correlation; (c) illustrates common case of moderate positive correlation.

(3) Figure 4-4(c) illustrates a more common case. The trend is toward increasing exterior channel stage as the interior runoff increases. However, high interior runoff is coincident with low exterior stage from time to time. Likewise, low interior runoff often is coincident with low exterior stage, but infrequent combinations with higher exterior stage do occur.

(4) The degree of correlation of coincident events may be quantified with the correlation coefficient. This statistic, which is commonly denoted *R*, is computed as described in EM 1110-2-1415. For that calculation, coincident observed values of interior runoff and exterior stage are used as the independent and dependent variables. The absolute value of *R* varies from zero to one, indicating the "fit" of the points to a trend line. A positive value of *R* indicates that the trend is for coincident stage to increase as discharge increases, while a negative value of *R* indicates that coincident stage is likely to decrease as discharge increases. For example, a computed value of *R* equals 1.0, this indicates that high exterior stage will occur coincident with high interior discharge, as in Figure 4-4(a). If any single value of stage is as likely as another for a given interior runoff (as illustrated in Figure 4-4(b)), *R* will approach 0.0.

e. Independent. Independent (or independence) is used herein to characterize the relationship between discharge and stage in an interior drainage system. Specifically, independent describes the situation in which discharge and/or stage at one location is not related to discharge and/or stage at another.

(1) Independence may be due to a lack of significant physical connection of conditions at the locations of interest. For example, in Figure 4-2, Location D is far upstream in the interior channel, out of the backwater of the pond. Thus stage at Location D is independent physically of stage in the pond at Location C and of the stage in the exterior channel at Location B. However, stage at Location D will be influenced by runoff from the interior watershed (A). Thus, stage at Location D is dependent on interior discharge.

(2) Independence may also describe a lack of statistical correlation. The situation illustrated by Figure 4-4 (b) is one in which the exterior stage and interior runoff are independent. The exterior stage does not appear to be in any way reliably related to or predicted by the interior discharge, and vice versa; the exterior stage and interior runoff are independent.

(3) Stage and discharge conditions in an interior watershed may be physically independent, yet exhibit great statistical correlation. For example, the stage at Location D in Figure 4-2 may be physically independent of exterior stage due to the lack of strong hydraulic connection. However, if the watersheds contributing to discharge in the interior area and exterior channel are subject to similar and coincident precipitation events, the stages may be highly correlated. The analyst must take care to avoid confusing statistical independence and physical independence.

(4) The degree of dependence is determined based on inspection of the available record and judgments with regard to the meteorologic and physiographic origins of the interior and exterior events. Context needs to be carefully defined; the fact that storms occur only in the winter (spring, etc.) is not an adequate basis for declaring that the occurrences are dependent. The critical focus must be on the aspects of the occurrences related to possible coincidence, since this is the critical item with respect to analysis. The validity of the assumptions necessary for application of the coincident frequency method is controlled by whether or not independence is the case.

4-3. Procedure Overview.

a. General. Two basic hydrologic procedures for analyzing with and without interior project conditions are presented. These approaches are continuous record analysis methods and coincident frequency methods. These procedures are summarized in paragraphs 4-3b and 4-3c, and described in detail in Sections 4-5 through 4-7.

b. Continuous Record Analysis. A frequency analysis of a continuous record develops the desired probability distribution of stage at the location of interest in the interior area, such as Location C in Figure 4-1 and Figure 4-2. The result is a stage-probability function that defines $P[Z_C]$ directly, where Z_C equals a selected value of stage at location C; and $P[Z_C]$ equals the probability that the annual maximum stage equals or exceeds Z_C. (Throughout this chapter, P[ ] is used to designate both probability of occurrence and probability of exceedance.) Continuous record procedures can be subcategorized as 1) period-of-record (historic) and 2) multiple discrete events of historic record. Analysis of multiple discrete events are included as a continuous analysis method since events relating to coincident flooding of local runoff and river stages are identified from historic record of river stages, interior stages, and rainfall. Each of the techniques may be used to develop hydrologic data of coincident flooding adjacent to the line-of-protection.

c. Coincident-Frequency Analysis. A coincident-frequency analysis develops the desired probability distribution of stage at the pond location from a flow-probability curve for the interior runoff and a duration-based probability distribution of the exterior stage. These probability distributions for the interior runoff and exterior stage are considered together using conditional probability. Conditional probability refers to the probability of occurrence (or exceedance) of one state of a property of the system, given a specified state of a second property upon which the first depends. For example, for the interior system shown in Figure 4-1, the interior stage at location C, Z_C, depends upon the interior discharge, Q_A and the coincident exterior stage, Z_B. If the value of Z_B is specified or fixed, the probability of various values of Z_C can be estimated with methods and models described in Section 4-7, considering only variation in Q_A. The resulting probability estimates are conditional probability estimates; that are "conditioned" on occurrence of the specified state of the exterior stage. This commonly is denoted $P[Z_C|Z_B]$ in which $Z_C|Z_B$ denotes occurrence of a value of Z_C given a specified value of Z_B.

(1) Figure 4-5 shows two stage-probability curves for the pond displayed in Figure 4-1. The same interior runoff is considered in both, but the exterior stage is different, as shown in figure 4.5. The upper frequency curve (the solid line) represents the interior pond stage for a specified value of the exterior stage - one that is high enough that the flap gate on the outlet is closed. When closed, the pond does not drain freely, so runoff from the interior area creates a greater stage in the pond. The lower frequency curve (the dash-dotted line) on the other hand represents the pond stage given a lower exterior stage in which the pond can drain freely. The frequency curves are conditional stage-probability curves, and the probability estimates for a specified pond stage are conditional probability estimates.

(2) Determination of the probability of pond stage is simplified somewhat when the exterior stage is fixed, as described above. In that case, $P[Z_C|Z_B]$ varies with interior runoff only. Thus, $P[Z_C|Z_B]$ takes on the probability of the value of Q_A that caused Z_C, given the value Z_B.

(3) The coincident-frequency analysis method, described in detail in Section 4-7, is an application of the total probability method. Total probability refers to the probability of occurrence (or exceedance) of a specified condition within the system, considering all possible combinations of contributing conditions. For example, interior pond elevation Z_C in Figure 4-1 (Location C) depends upon the exterior stage Z_B. Thus, the probability of exceeding a specified elevation Z_C depends on the probability of the coincident exterior stage. To assess the true (or total) probability of exceeding a specified interior stage, the likelihood of all possible exterior stages must be considered.

d. In many cases, observed flow and/or stage data will not be available to perform the analysis described in sections 4-3b and 4-3c. In these cases, hydrologic models can be developed for the interior area and used to compute a continuous record of flow from the interior drainage area. Refer to EM 1110-2-1417 for guidance on the development of a precipitation-runoff model in ungaged watersheds. Exterior stage data can be developed using a river hydraulics model or hydrologic routing. The routing model would begin at a gaged location upstream of the interior area outlet and extend to a gage downstream of the interior area outlet. The routing model would be calibrated to flood events and then a period-of-record simulation would be simulated to determine flow and stage at the interior area outlet.

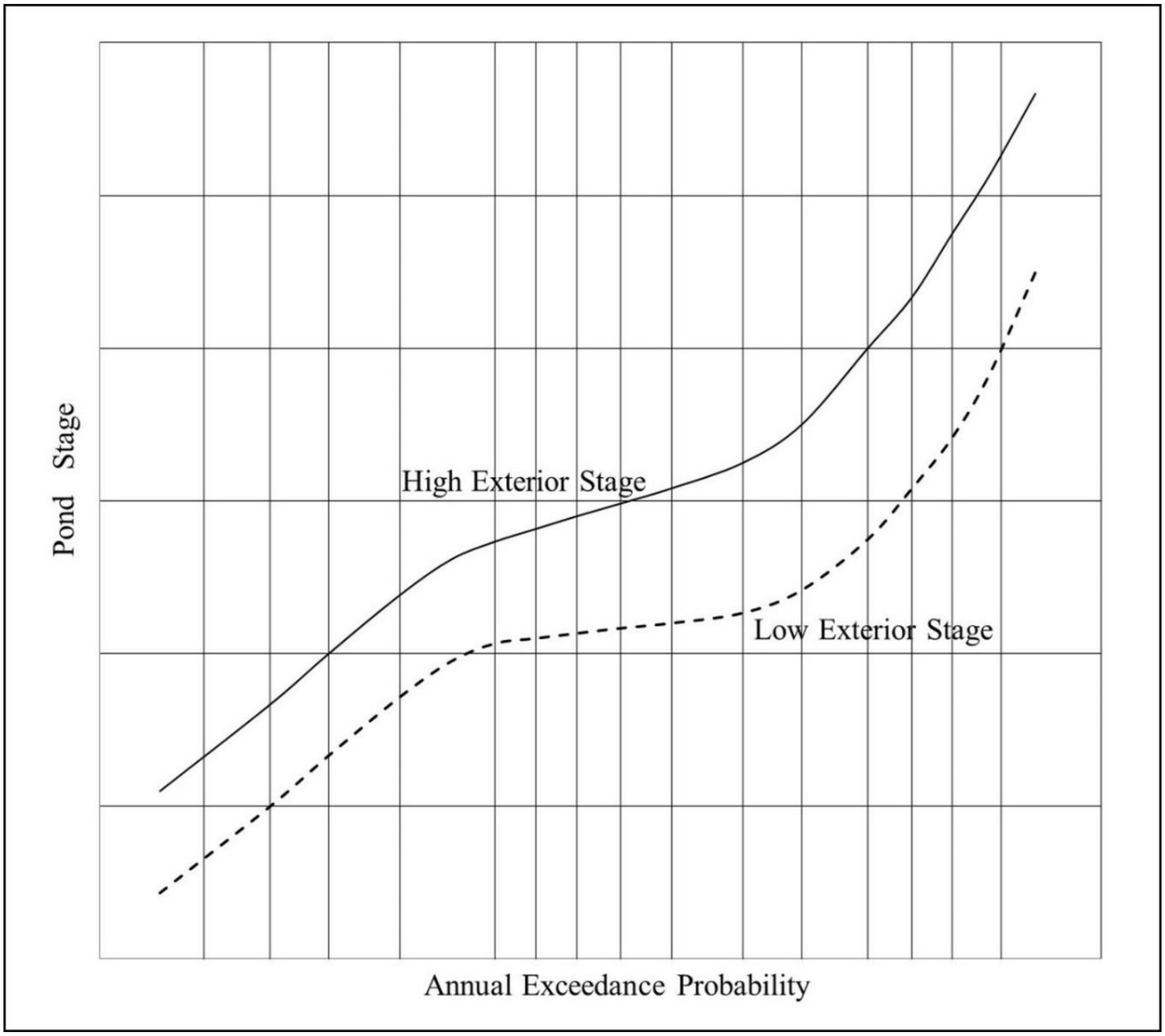

Figure 4-5. Illustration of conditional probability estimates of interior stage: each curve represents the probability of pond stage given a certain exterior stage.

4-4. Hydrologic Data Requirements.

a. General. Hydrologic data required for analysis of interior areas can include: annual maximum stages of interior pond, topography, exterior stage data, interior runoff data, historic rainfall records, evapotranspiration data, frequency precipitation data for the interior watershed, hydrologic modeling parameters, and seepage data. Physical characteristics of the pond and operation procedures for the without condition, must also be determined. Refer to EM 1110-2-1417 for guidance on the development of a calibrated and validated precipitation-runoff model and application of the model to frequency based hypothetical events.

b. Annual Maximum Stages. If a long term sample of stages is available at the interior pond, a stage-probability curve can be developed from the data. Care must be taken that the interior stage record represents a wide range of interior and exterior stages.

c. Topography. Topographic data are required to define watershed and subbasin boundaries, runoff parameters (slopes, stream lengths), and estimation of elevation-area-storage relationships for natural detention areas.

d. Exterior Stage Data. Exterior stage data are required primarily at gravity and pumping station outlet locations (exterior flow data along with a hydraulics model could be used in place of stage data). Secondary gravity outlet data may be aggregated (combined rating curves) to primary outlet locations, or ignored if the discharge capacity is insignificant relative to the primary outlets. The exterior stage data should be a time-series of stage for the period-of-record. If no stage data is available at the outlet location then a hydraulics model could be used to route the stage hydrograph from an upstream gaged location to the outlet.

e. Interior Runoff Data. Interior runoff data includes measured inflows into the ponded area and at upstream locations in the watershed. An inflow-probability relationship can be developed directly when the sample of historic pond inflows is long enough. Otherwise, the interior runoff data are needed to calibrate and validate the rainfall-runoff model of the interior watershed. Interior runoff can be used to assess correlation to exterior stage.

f. Rainfall Data. Rainfall data are required for developing rainfall-runoff models for the interior and possibly for exterior areas. The data should be for the period-of-record and at a time-resolution appropriate for the interior watershed. If no rainfall gage exists within the watershed, records from nearby rain gages will be used in the analysis. Precipitation on the interior watershed can be used to assess correlation to exterior stage.

g. Runoff Parameters. Hydrologic parameters for rainfall-runoff modeling are required for loss rates, runoff transforms, base flow, and routing. Loss rate parameters may be initially estimated by using values from previous studies, or derived through analysis of measured rainfall and runoff volumes at gages. Loss rates are generally based on the land use, antecedent soil moisture condition, and physical basin characteristics. Initial values for unit hydrograph and other runoff transform parameters may be estimated from land use and physical basin characteristics using published values or regression equations. The importance of runoff volume, rather than peak discharge, permits the use of simplified modeling methods to be employed with acceptable results. Calibration studies of assumptions and verification of results to high water marks and frequency information must be performed as needed.

h. Physical and Operational Characteristics of Existing Measures. Information on physical and operational characteristics of existing flood loss reduction measures is normally required. Gravity outlet locations, capacity, and operation procedures are needed to enable simulation analysis to reproduce the historic record.

i. Other Data. Data on ponding areas, collection systems (storm sewers), and any hydraulic controls effecting water movement are also often necessary.

4-5. Continuous Record – Period-of-Record Analysis. Period-of-record methods involve analysis of continuous historic records of hydrologic data. In some cases, a continuous record of the interior stage is available. In most cases, the interior stage will need to be computed using measured or modeled discharge from the interior area and exterior stage. The procedure for this analysis consists of performing continuous hydrologic simulation of inflow, outflow, and change in storage to derive interior water surface elevations (Location C, Figure 4-1) given exterior stages and interior runoff for the entire period-of-record. A diagram of this procedure is shown in Figure 4-6.

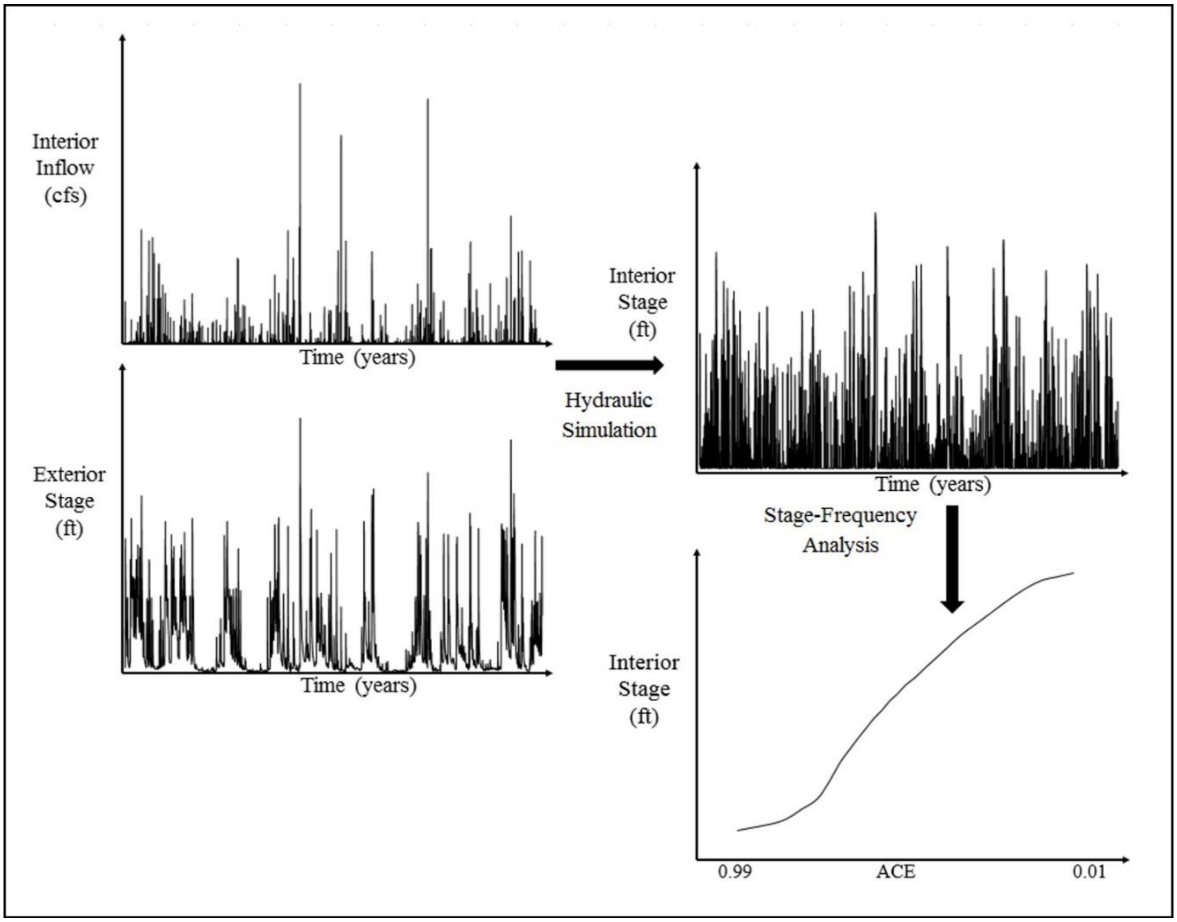

Figure 4-6. Diagram of period-of-record analysis procedure.

a. General. In some cases, observed data is available for the interior watershed. The interior runoff data could be available directly at the interior pond or upstream of the pond. When located upstream of the pond, a relationship between drainage area, and some other hydrologic variable could be used to estimate the inflow directly into the pond. In other cases, no discharge from the interior area will be available. In this case, historic precipitation data is typically applied to a calibrated hydrology model to yield runoff hydrographs at subbasin outlets. Hydrographs are combined and routed through the system (as appropriate) to gravity outlets and pumping stations to yield period-of-record inflows at the line-of-protection.

(1) The period-of-record procedure is attractive because it preserves the seasonality, persistence, and dependence or independence of exterior (river) stages and interior flooding. The method enables the performance of the project to be displayed in a manner easily understood by the other study participants and the public. The procedure is particularly useful for evaluating crop damage of single subbasin watersheds (ponding adjacent to line-of-protection) in agricultural areas. System operational and maintenance costs may be calculated directly. The methods are generally tedious to apply because of the large amount of hydrologic data analyzed.

(2) Major considerations in application of the period-of-record procedures are the potential for the historic record being unrepresentative (records are usually short), and that the procedure requires significant information needs and extensive calibration. A short and unrepresentative historic record may yield inappropriate size and mix of measures and operation specifications of the system. The extensive data needs and model calibration requirements often result in a period-of-record analysis that is an unduly simplistic rainfall-runoff analysis for single subbasins adjacent to the line-of protection. The level of detail is often adequate for agricultural areas, but may not be for the runoff-routing analyses required of complex urban areas.

b. Data Requirements. Data and information required for synthesis of an interior stage time-series include the properties of the interior watershed and interior channels, including (but not limited to) information shown in Table 4-2. With this information, models of the watershed runoff and channel flow will be configured and parameters of the models estimated.

Table 4-2. Properties of the interior watershed and interior channels required for model configuration and calibration.

Property	Use
Topographic, terrain data	Delineate interior watershed (including interior area storage), identify drainage patterns, define subbasins for rainfall-runoff model, and estimate overland flow model parameters.
Land use information and soil data	Estimate parameters of watershed infiltration and overland flow models.
Locations, dimensions, and other properties of conveyance, including properties of channels, conduits, storm water management facilities	Configure and estimate parameters of models of conveyance of water (routing) from interior watershed to facilities at line-of-protection. Also, configure and calibrate models with which stage can be estimated, given discharge (and vice versa).
Locations, dimensions, operation procedures for interior area storage, or diversion for flood risk reduction upstream of the line-of-protection	Configure and estimate parameters of models of these facilities to capture impacts on discharge to line-of-protection facilities.

(1) Historical discharge time-series for the interior watershed. This could be available directly at the interior pond or upstream of the pond. When located upstream of the pond, a relationship between drainage area and some other hydrologic variable could be used to estimate the inflow directly into the pond.

(2) If historic measurements of discharge from the interior watershed are not available then a rainfall-runoff model can be used to develop the continuous discharge hydrograph using historical precipitation and evapotranspiration. The precipitation data can come from observations at gages in or near the interior watershed and from radar estimated precipitation. The desired record is a continuous time-series for a reasonably long period during which both precipitation and coincident exterior stage are observed. The precipitation data is used for computation of interior area runoff hydrographs for a long period.

(3) Properties of facilities at the line-of-protection. This information includes locations, dimensions, and operating procedures for gravity outlets, pumps, and detention ponds. If seepage at the line-of-protection contributes in a significant manner to water that must be managed at the line-of-protection, information on seepage rates and influences must also be collected. With all the information, a simulation model of the facilities at the line-of-protection is developed. That model is coupled with the model of interior runoff and information about exterior conditions to compute required stage at the line-of-protection, thus, synthesizing the necessary record for frequency analysis.

(4) Information about exterior conditions. Ideally, this information includes a historical series of discharge or stage in the exterior channel at the outlet of the interior drainage facilities. Exterior stage time-series data were in most cases derived from discharge data from the nearest upstream or downstream gage. Historical flows from that gage will be adjusted or routed as necessary to account for intervening flows and travel time. Those flows will in turn be converted back to stage at the project location by means of a rating function. The time step of the exterior flow data should be of adequate resolution to capture the rising and falling of exterior stages. If no observed time-series of exterior stage are available, properties of the exterior channel and upstream watershed are needed to configure and calibrate runoff and routing models from which a series of stage in the exterior channel can be synthesized in much the same way as a series of interior runoff and stage is synthesized.

(5) Historical observations of stage, discharge, and flooding in the interior area for model calibration and validation. This includes available interior area streamflow records, records of pond stages, pumping rates, gravity drain flows, and reports of flooding in the interior area (such as high water marks).

c. Analysis Steps. The following analysis procedure is for the case when no time-series of observed interior runoff is available. In this case, historic precipitation is applied to a rainfall-runoff model. It is also assumed that the exterior stage time-series already exists at the outlet of the interior drainage facilities. Figure 4-7 shows a schematic of this procedure.

(1) Watershed and subbasin boundaries are delineated and damage reach index locations selected where hydrologic data are developed for flood damage analysis.

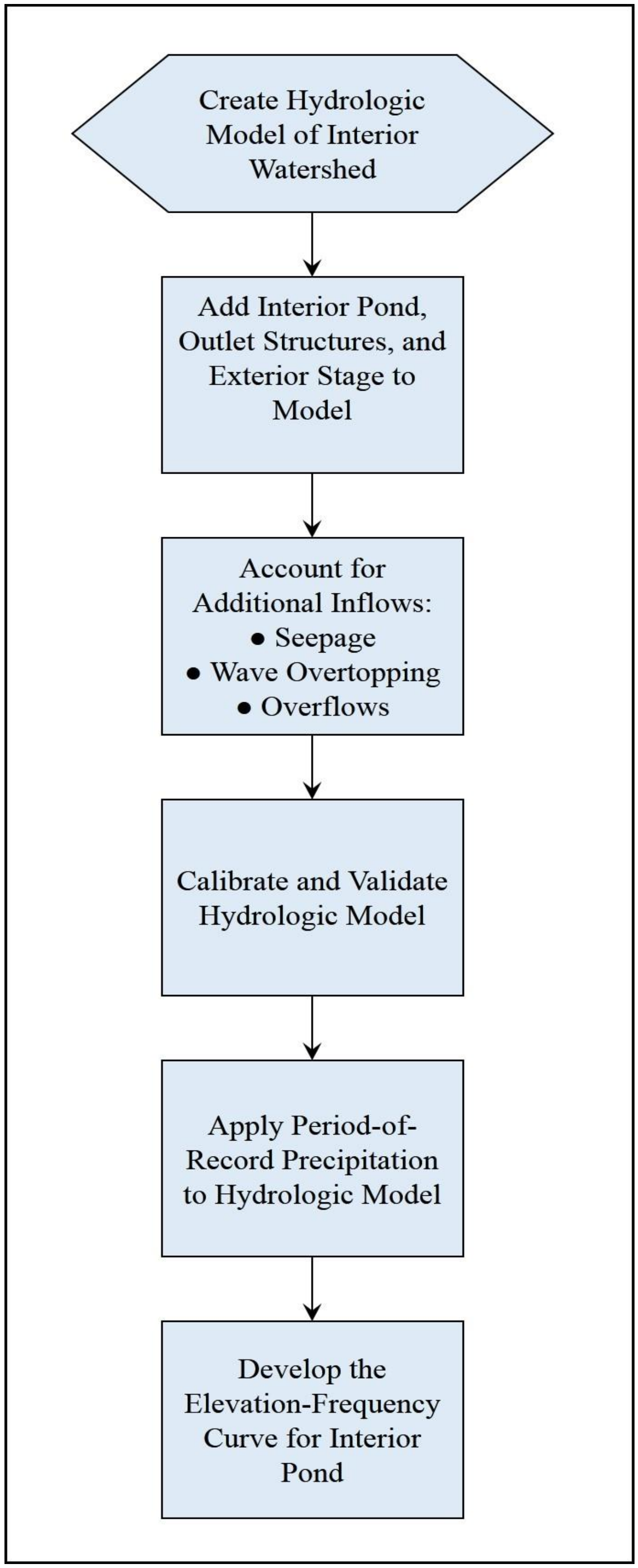

Figure 4-7. Schematic of period-of-record analysis steps.

(2) Configure and calibrate the interior watershed runoff and channel models, using the information collected, and confirm proper functioning of the models with available observations.

(3) Other contributing interior flows such as seepage, wave overtopping, and overflow from adjacent areas are determined for use in the analysis.

(4) With the period of record precipitation, simulate watershed runoff and discharge into the facilities at the line-of-protection. Using the coincident exterior stage, compute the interior stage series at the location of interest, adding seepage, water contributed by wave or wind-driven overtopping, and other sources as appropriate.

(5) Develop the elevation-frequency relationship for the interior stage (using the annual maximum stage from the computed results), duration of flooding, and other pertinent hydrologic information at locations of interest for the existing without-project conditions. Refer to EM 1110-2-1415 for guidance on developing stage-probability curves. As stated in EM 1110-2-1415, elevation-frequency curves are typically fit manually to the stage data as analytical frequency curves do not fit the stage data adequately.

(6) Examine the resulting frequency curve to ensure that the results are consistent regionally with similar curves for other sites and that stage or discharge quantiles (values for a selected probability) are reasonable, given observations at-site. Consider, for example, observed flooding in the interior area and the likelihood of that experience on the context of the predicted frequency of flooding with the frequency curve.

(7) Repeat Steps 2 through 6 for other conditions within the interior system as needed. For example, if an interior stage-probability function is required to reflect the impact of future land use in the interior watershed, adjust the hydrologic model parameters in Step 2 and repeat the calculations, using the same precipitation and coincident exterior stage record as before.

(8) For locations where different runoff generating mechanisms can cause flood flows in the interior area, like tropical and non-tropical floods, a mixed population analysis could be performed. Step 4 would be modified to separate the storm events into their storm type. Separate stage-frequency curves would be developed for each flood type and then the stage-frequency curves would be combined as discussed in EM 1110-2-1415.

d. Software. No specific software applications are required for interior area analysis for USACE projects. However a number of applications, such as those listed in Table 4-3, can assist in the analysis with the computations identified. Other programs can be used as needed if those programs are accepted for use in USACE studies.

Table 4-3. Software applications that may be useful for interior analysis.

Software application	Use
HEC-HMS Hydrologic Modeling System	Compute watershed runoff from precipitation; route hydrographs to line-of-protection with simplified channel models; simulate behavior of interior storage and diversion; simulate behavior of pumps and gravity drains with simplified model. Developed by USACE, Hydrologic Engineering Center (CEIWR-HEC).
HEC-RAS River Analysis System	Route hydrographs to line-of-protection with dynamic wave model; simulate behavior of pumps and gravity drains; develop stage-discharge transforms needed. Developed by USACE, Hydrologic Engineering Center (CEIWR-HEC).
HEC-SSP Statistical Software Package	Analyze time-series to estimate parameters for fitting frequency curves; can also be used to fit stage-probability curves graphically and aid in the computation of a coincident frequency analysis. Developed by USACE, Hydrologic Engineering Center (CEIWR-HEC).
HEC-FDA Flood Damage Reduction Analysis	HEC-FDA is used for determining EAD and performance indices for project alternatives. HEC-FDA is certified software, and such is appropriate for this analysis. EM 1110-2-1619 and the HEC-FDA User's Manual contains detailed descriptions of data needed for HEC-FDA (USACE, 2015). Developed by USACE, Hydrologic Engineering Center (CEIWR-HEC).
GSSHA Gridded Surface/Subsurface Hydrologic Analysis	Surface water and watershed modeling software for computing basin runoff and river stage. Developed by USACE, Engineer Research and Development Center (CEERDC).

4-6. Continuous Record - Multiple Discrete Events. The multiple discrete event procedure is based on development of interior stage-probability functions for areas affected by coincident flooding. The procedure generates a composite stage-probability function from analysis of two conditions. The first involves analysis of selected (high stage) exterior events of historic record that have an effect on interior flooding when interior rainfall occurs coincidently. The second condition involves analyses of low exterior stages associated with interior flood analysis generated by either coincident historic rainfall or hypothetical frequency storm events. For the second condition, historic rainfall is commonly used in agricultural areas and hypothetical frequency rainfall for analysis of urban areas. The result is a stage-probability function for each of the two conditions. They are then combined into a composite function by the application of the joint probability theorem. A diagram of this procedure is shown in Figure 4-8.

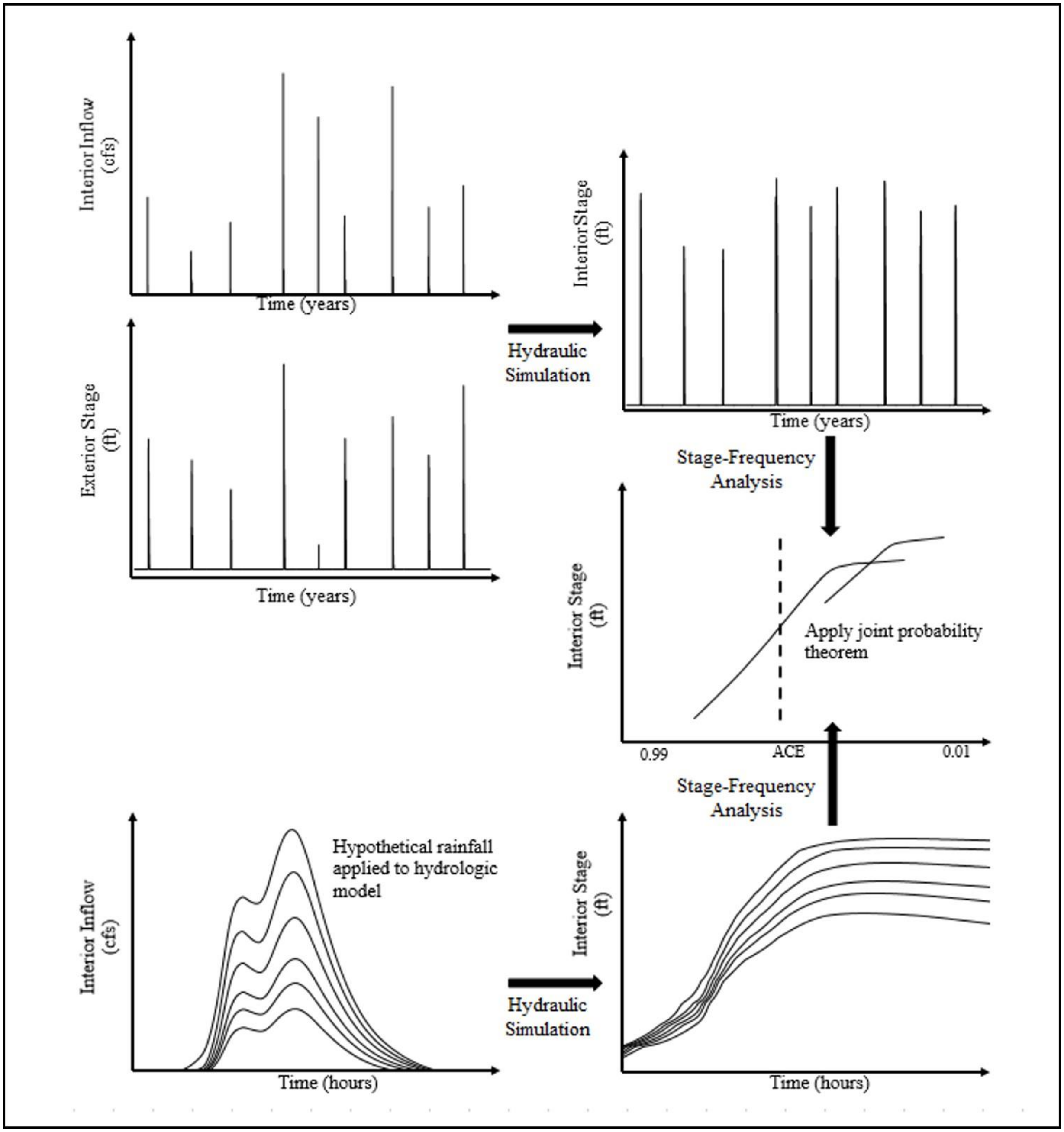

Figure 4-8. Diagram of multiple discrete events analysis procedure.

a. General. The multiple discrete event method is similar to the period-of-record procedure in that the concepts of coincident flood simulation are easy to understand and antecedent moisture conditions are accountable. Both methods may be influenced by short and unrepresentative historic records. The two procedures are different in that the discrete event analysis evaluates fewer events, uses fewer parameters, and generally is more applicable for complex hydrologic systems. Combining probability functions is a distinct departure as well.

The discrete event method may miss events that impact the results, and does result in a less automated process of analysis than the period-of-record.

b. Data Requirements. Data and information required for the multiple discrete events analysis is similar to that required by the period-of-record analysis. The list of data can be found in Section 4-5b. Additional data requirements include hypothetical precipitation for a range of frequencies.

c. Analysis Steps. The hydrologic procedures typically applied to perform multiple discrete analyses of interior areas are shown in Figure 4-9.

(1) The historic record of exterior stages is reviewed to determine the events which may have an impact on interior flooding. Dividing the record by season may be an important consideration. Unless seepage or overflow from adjacent areas or wave overtopping becomes significant problems, events must occur coincidently with interior events that result in damage when the gravity outlets are closed. The event definition should identify dates, be of sufficient length to determine duration and seasonal effects on the damage potential, and assess antecedent moisture conditions.

(2) Rainfall-runoff and interior routing procedures for high exterior stage events are similar to those described for the period-of-record analysis, except evaluations are performed for single historic events. Historic rainfall data must be coincident with the exterior events selected for the analysis. Rainfall excess is applied to runoff transforms and routed to produce hydrographs throughout the interior system. Seepage and other inflow functions are developed. Total hydrographs are subsequently routed through existing gravity outlets and pumping stations. The gravity outlets are blocked until a positive differential head exists between the interior and exterior.

(3) Stage-probability functions are developed using results from the historic events (high exterior stage events). The events are normally ranked in decreasing order and plotting positions established based on the historic record length.

(4) Analysis of low exterior stage events normally use hypothetical frequency storm and runoff analyses for urban areas and historic events for agricultural crop damage assessments. If historic events are used, maximum intensity rainfall is selected from continuous records for the period coincident with low exterior stage.

(5) Rainfall-runoff analyses are performed for the low exterior stage events. To compute interior stage, hydrographs are routed through the line-of-protection assuming low exterior stage conditions. Stage-probability relationships are developed at desired locations. If hypothetical frequency storms are used, the frequency functions can be developed directly from the recurrence functions. Refer to EM 1110-2-1417 for developing hypothetical storms and assigning exceedance probabilities to the computed flow. For historic storms, the events are ranked and plotting positions assigned.

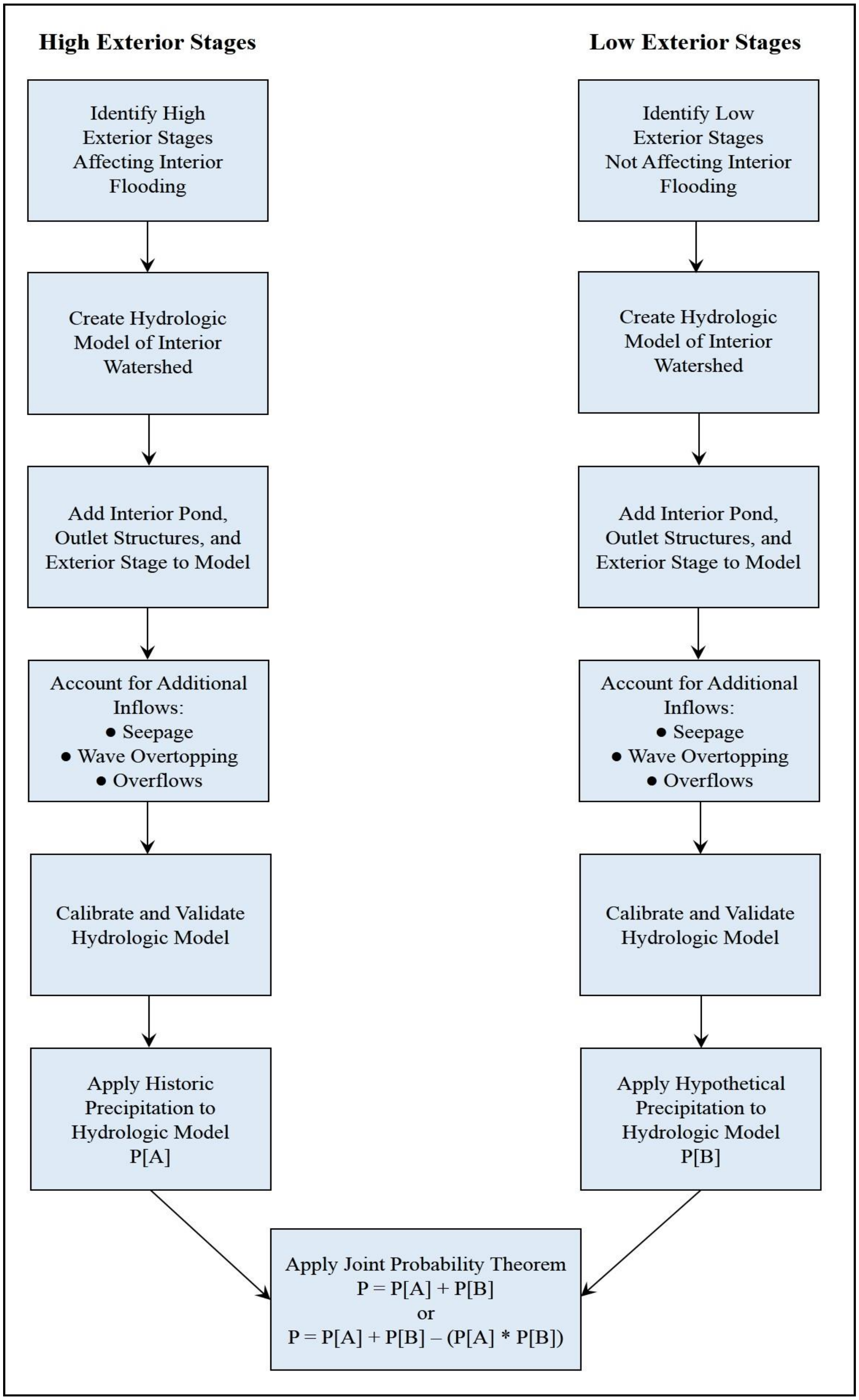

Figure 4-9. Schematic of multiple discrete events analysis steps.

(6) The joint probability theorem is used to combine the frequency functions for high and low exterior stage conditions. For annual series, total probability is equal to the sum of the probability at that stage (or flow) for each relationship minus the product of their individual probabilities (to subtract probability of events occurring in the same year). For partial series (with multiple events in a year, assumed to cause damage), the total probability is the sum of the probability at that stage (or flow) for each relationship.

(7) Repeat Steps 2 through 6 for other conditions within the interior system as needed. For example, if an interior stage-probability function is required to reflect the impact of future land use in the interior watershed, adjust the hydrologic model parameters in Step 2 and repeat the calculations, using the same precipitation and coincident exterior stage record as before.

d. Software. No specific software applications are required for interior area analysis for USACE projects. However, a number of applications, such as those listed in Table 4-3, can assist in the analysis with the computations identified. Other programs can be used as needed if those programs are accepted for use in USACE studies.

4-7. <u>Coincident Frequency Analysis</u>.

a. Overview. The term coincident frequency analysis is used to describe the category of analysis procedures that derive the required interior stage-probability function from frequency functions of the contributing states of the interior system. In this case, the contributing states are runoff from the interior watershed and stage in the exterior river. The frequency functions of contributing states are developed with standard procedures described below.

(1) The coincident frequency analysis method described in this document is an application of the total probability method. That method uses the total probability theorem to derive the interior stage-probability function as a function of the interior watershed discharge frequency and the coincident exterior stage-probability. The total probability theorem, applied to the case of interior flooding illustrated by Figure 4-1, defines $P[Z_C]$, the probability of occurrence or exceedance of an interior stage, Z_C, at Location C. This computation considers all possible coincident exterior stages, Z_B, partitioning the range of all possible exterior stages into mutually exclusive values. $P[Z_{Bi}]$ equals the probability associated with exterior stage Z_{Bi}; and $P[Z_C|Z_{Bi}]$ equals the conditional probability of the interior stage Z_C given exterior stage Z_{Bi}.

$$P\left[Z_C\right] = \sum_{i=1}^{n} P\left[Z_C \mid Z_{Bi}\right] * P\left[Z_{Bi}\right] \qquad (4\text{-}1)$$

(2) In certain cases, application of Equation 4-1 is simplified. If interior and exterior conditions are correlated to the extent that a certain exterior stage always is coincident with a given interior stage, only that single case need be considered in the calculations. For example, if analysis of the historical record indicates that an exterior stage with exceedance probability 0.02 always coincides with an interior stage of probability 0.01, only that coincident combination

need be considered in the computations. All other conditional probability values $P[Z_C|Z_{Bi}]$ equals 0.00 (or nearly so), as no other coincident combinations are likely. This may be the case, for example, if large regional events commonly cause large interior discharges at the same time that exterior stages are great.

(3) For those cases in which various interior and exterior coincident conditions occur in a less predictable manner, application of the total probability method considers all possible combinations and their likelihood. Application in that case is illustrated with frequency curves shown in Figure 4-10. In this simple example, the entire range of possible exterior events is considered well represented with just two mutually exclusive but exhaustive cases: a high exterior stage and a low exterior stage. Various interior events can occur coincident with either case. For the high exterior case, the interior pond stage-probability function is developed as

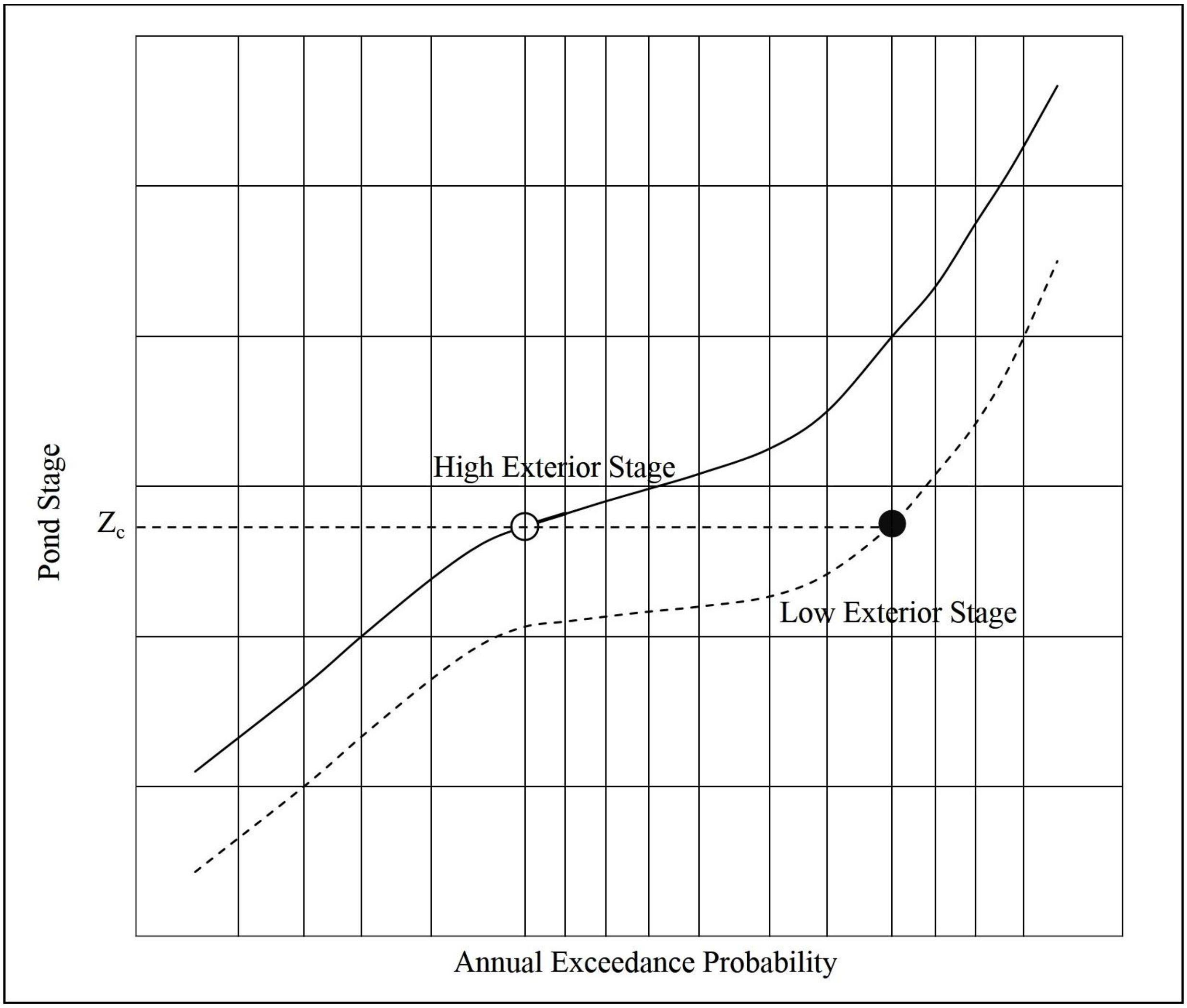

Figure 4-10. Illustration of application of total probability method: Total probability of Z_C is computed from conditional probability of Z_C with high exterior stage and conditional probability of Z_C with low exterior stage in this example.

described in Section 4-7c. That conditional-frequency function is represented with the solid line in Figure 4-10. Similarly, the conditional interior pond stage-probability function for low exterior stage is developed; that is represented by the dot-dashed line in Figure 4-10. To estimate the total probability of a specified interior stage, Z_C, with the total probability equation (Equation 4-1), the probability of Z_C given coincidence with high exterior stage is found; that point on the frequency curve is represented by the open circle in Figure 4-10. This probability is multiplied by the probability of high exterior stage coincident with the value Z_C. This result is added to the product of the probability of the same interior stage, given low exterior stage (denoted with the filled circle, Figure 4-10) and the probability of low exterior stage. This presumes that the interior and exterior events are independent. The coincident frequency analysis example in Chapter 9 contains a sample calculation for how the probability of a given stage is computed from multiple conditional frequency functions.

(4) For the case illustrated by Figure 4-10, the probability of any pond stage for a specified exterior case takes on the value of the probability of the interior runoff that creates that pond stage. That is, $P[Z_C|\text{high } Z_B] = P[Q_A \text{ that led to } Z_C]$. For example, the conditional pond stage with probability is equal to 0.01, which is caused when the runoff peak discharge with exceedance probability is equal to 0.01.

b. Data Requirements. Data and information required for a coincident frequency analysis are similar to those in the continuous record analysis. In many cases, a hydrologic model of the interior watershed will be needed to generate runoff hydrographs for specific probabilities. Simulation of interior runoff requires the properties of the interior watershed and interior channels, including (but not limited to) information shown in Table 4-2. With this information, models of the watershed runoff and channel flow will be configured and parameters of the models estimated. Runoff hydrographs from the interior watershed, along with physical properties of the pond and exterior stage, are used to compute the interior stage.

(1) Sufficient data on interior and coincident exterior conditions to determine if the conditions are independent, highly dependent, or exhibit moderate correlation. Ideally, these data will include observed interior runoff peaks and observed exterior stages that are coincident. If measurements of interior runoff are not available, measured precipitation may be used as a surrogate, or a rainfall-runoff-routing model may be used to synthesize discharge for comparisons needed. If exterior stage measurements are not available at the outlet of the drainage facilities, measurements at a nearby location on the exterior channel may be used as a surrogate, or a hydraulic routing model can be used to synthesize exterior stage at the site.

(2) Models and data to determine the interior runoff-frequency function. If observed discharge data were available in the interior area, the discharge frequency function can be fitted with those data, following procedures described in EM 1110-2-1415. However, such data were rarely available. Instead, a more common approach to developing the required interior runoff-frequency function is to configure, calibrate, and apply a rainfall-runoff-routing model, using rainfall events of specified probability as forcing functions for the model, assigning probability to the computed discharge peaks consistent with the probability of the precipitation.

(3) Models and data determine the appropriate exterior stage duration curve. Again, the ideal situation is that measured stage records in the exterior channel at the outlet of the interior drainage facilities are available. If data were not available at the location of interest, but nearby, hydraulic routing models can be used to synthesize the required series. And if no appropriate data were available, a continuous rainfall-runoff-routing model can be used to generate a period-of-record, time-series of external stage.

(4) A model(s) of the interior drainage facilities, with which the pond stage can be computed, given the interior runoff and the exterior stage are consistent with requirements identified in Section 4-4, as the models provide much the same information. The difference in application is the forcing functions (or meteorologic conditions) used. For continuous record synthesis, the precipitation events analyzed are historical events, while for coincident frequency analysis, the precipitation forcing functions are storms of specified probability. Similarly, the model of outlet hydraulics used for synthesis of a long record will use observed exterior stages, while for coincident analysis, exterior stages of specified frequency are used. Nevertheless, the models are the same, representing current or future, without- or with-project conditions.

c. Analysis Steps. The coincident frequency approach utilizes a series of hypothetical single event hydrographs for the interior analysis and stage-duration (stage versus percent of time exceeded) for exterior stages. Basic steps in the approach are defined below and Figure 4-11 illustrates the general procedure.

(1) Delineate watershed subbasin boundaries and establish damage reach index locations where hydrologic data (discharge or elevation-frequency functions) are required. A stage-duration (or flow-duration) function is developed for exterior stages at primary outlet locations. The duration curve is typically developed using historic gaged data. The data were often transferred from a nearby gage. Adjustments may be needed if exterior stage differences between gage locations and study locations are significant. Figure 4-12 shows a duration curve for the exterior stage, in which stage is plotted against the percent of time that stage is exceeded. The duration curve is divided into discrete segments and the middle value of each segment is taken as an index river stage that takes on that segments probability. The duration curve in Figure 4-12 is divided into six segments. The segment interval, $P[Z_{B1}]$, zero to ten percent of the time exceeded represents the probability of the interval. The sum of the probabilities must equal 1, i.e., $\sum P[Z_{Bi}]$ equals one. In Figure 4-12, the probability assigned to Z_{B1}, Z_{B2}, Z_{B5}, and Z_{B6} is ten percent and the probability assigned to Z_{B3} and Z_{B4} is thirty percent. The segments are smaller at the tails of the duration curve in order to select an index value that is representative over the segment. More segments with smaller probability ranges are required as the slope of the duration curve becomes larger. The discrete representation should capture critical stages, including for example, stage at which gravity outlets close. If the exterior discharge is regulated, the discrete representation of the function should also be developed to capture significant shifts in stage that correspond to shifts in the upstream reservoir operation. For example, if upstream releases are "ramped up" at a certain frequency, the stage associated with that change in operation should be represented in the discretized representation of the function.

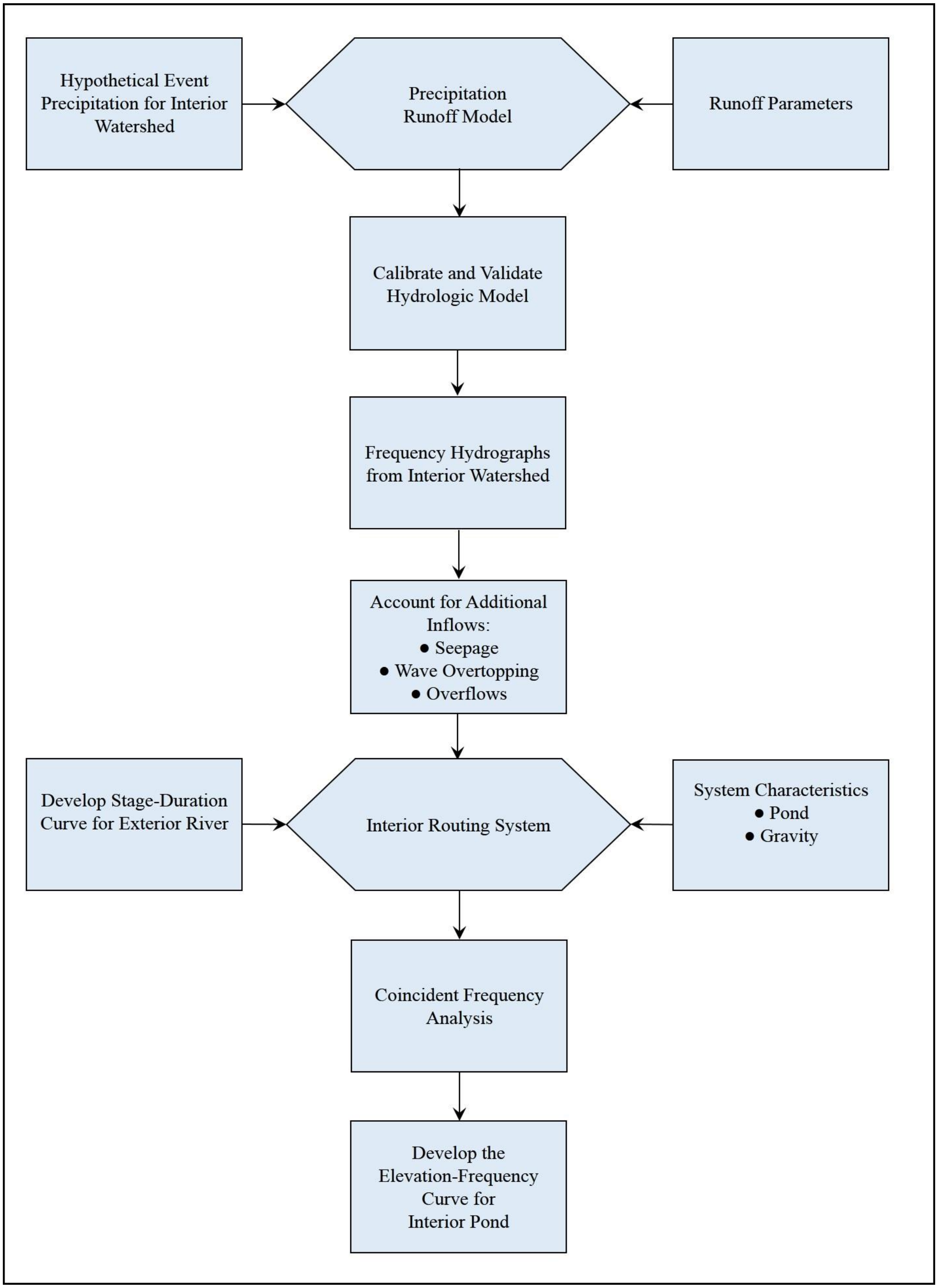

Figure 4-11. Schematic for a coincident frequency analysis.

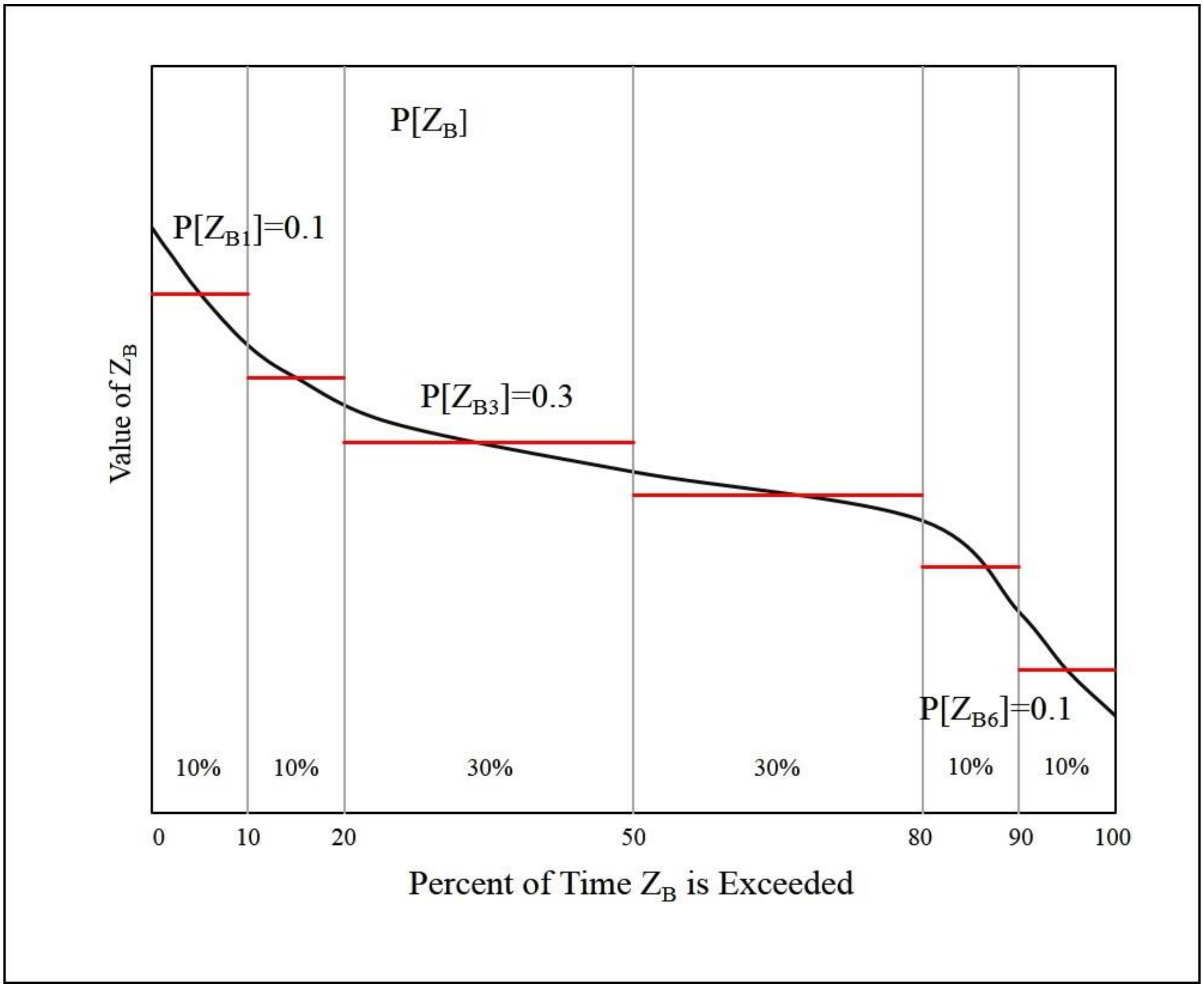

Figure 4-12. Duration curve for exterior stage.

(2) Define the runoff-frequency curve for the interior watershed, thereby permitting estimation of $P[Q_A]$ for discharge Q_A (an example is shown in Figure 4-13). This may be accomplished by fitting a probability distribution where streamflow data series were available and appropriate, or by using regional regression, or rainfall-runoff modeling using frequency precipitation (this last option will be the case in most studies). This step should be completed for all relevant conditions, including both without-project and with-project conditions (if proposed measures alter in any way the probability function) and current and future watershed conditions (if the most likely future condition will alter watershed runoff in any manner).

(3) Configure and calibrate a model of the interior facilities. This model will be used to compute interior stage given an interior runoff and an exterior stage.

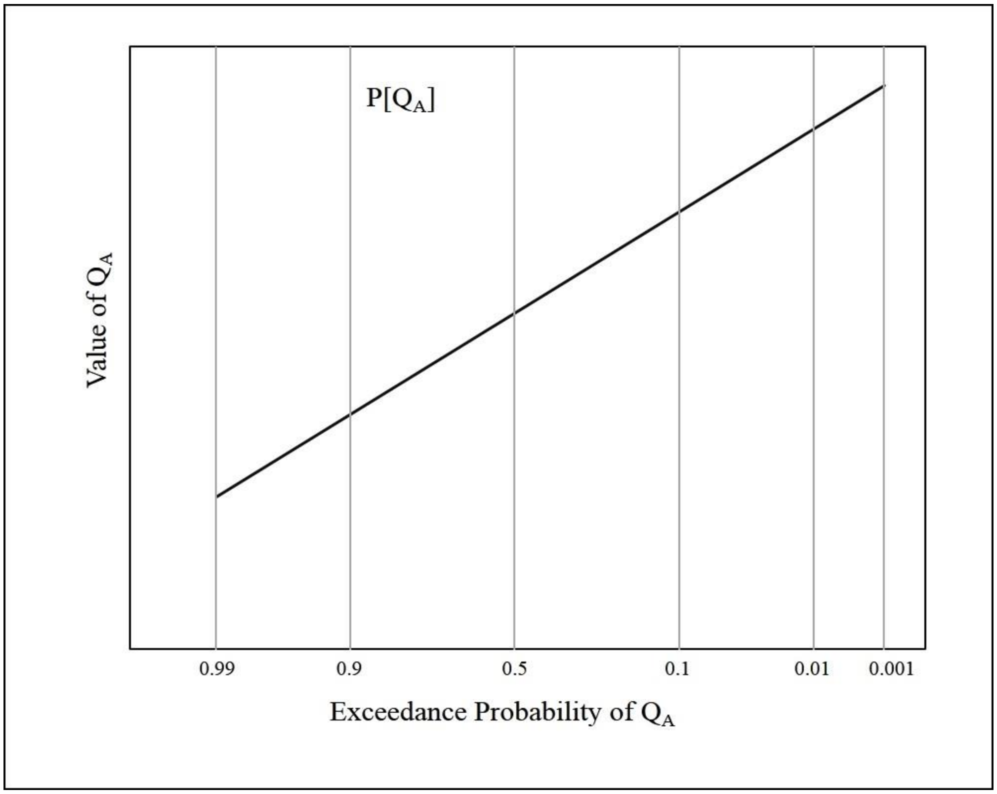

Figure 4-13. Runoff-probability curve for interior watershed.

(4) Select an exterior stage from the discretized duration function from Step 1.

(5) Select a discharge quantile Q_A (or a corresponding hydrograph with the same probability) from the interior watershed runoff-frequency curve, and identify the annual exceedance probability $P[Q_A]$. Use this quantile as the upstream boundary condition for the facilities model, with the exterior stage, to compute the interior stage. The probability of this stage, which is a conditional stage is the same as the probability of the interior runoff, $P[Q_A]$.

(6) Repeat Step 5 for the entire range of discharge values from the interior watershed runoff-frequency curve, keeping the exterior stage fixed. The resulting values of Z_C define the conditional interior stage-probability function for the specified exterior stage, $P[Z_C|Z_{Bi}]$.

(7) Return to Step 4 and select another exterior stage value from the discretized duration function and repeat Steps 5 and 6 to define another conditional interior stage-probability function. Repeat this for all index values defined for the exterior stage duration function. The computation of the interior stage given interior runoff and exterior stage will produce an array of conditional probability curves for interior stage as shown in Figure 4-14.

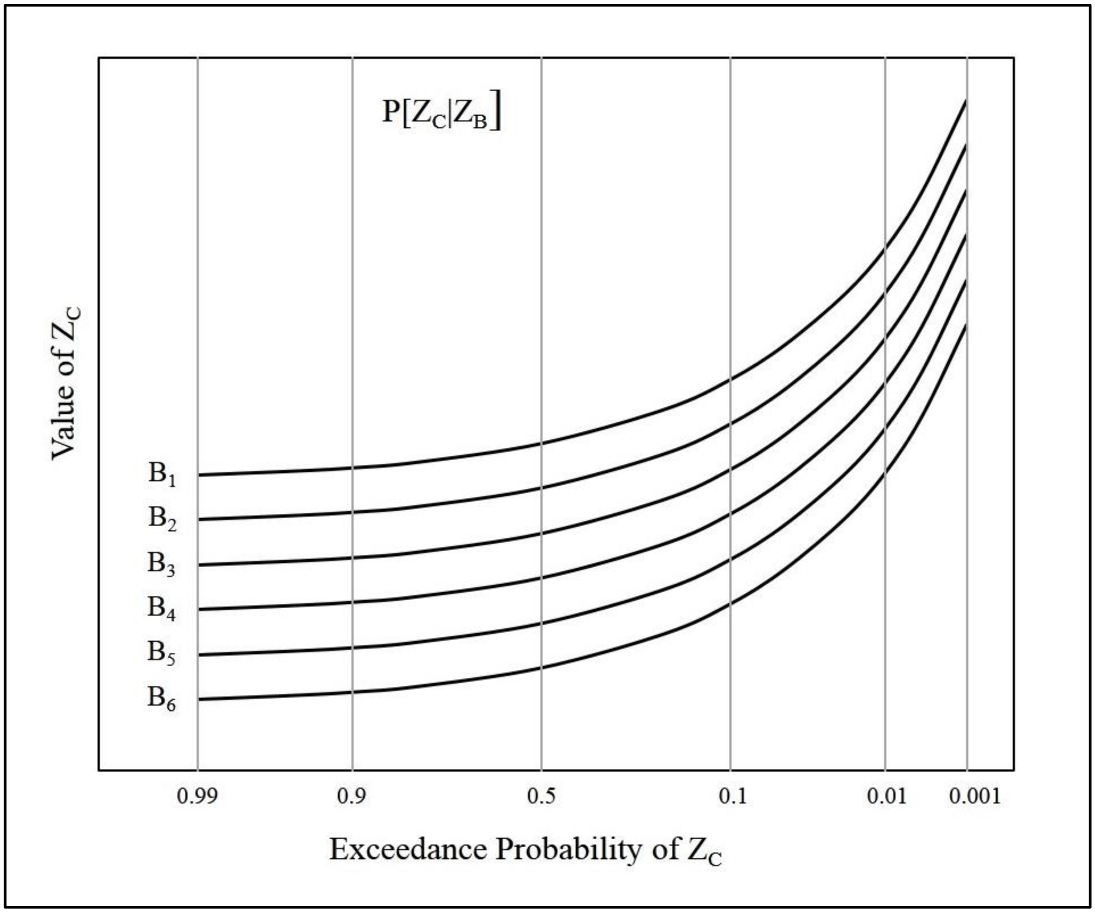

Figure 4-14. Conditional interior stage probability curves.

(8) When the entire range of exterior stage has been considered and all conditional stage-probability functions defined use the total probability equation (Equation 4-1) to compute the interior stage-probability function. Below is an example application of the total probability equation to compute one stage value on the interior stage-probability curve.

$$P[Z_{C1}] = P[Z_{C1}|Z_{B1}] \cdot P[Z_{B1}] + P[Z_{C1}|Z_{B2}] \cdot P[Z_{B2}] + P[Z_{C1}|Z_{B3}] \cdot P[Z_{B3}] + P[Z_{C1}|Z_{B4}] \cdot P[Z_{B4}] + P[Z_{C1}|Z_{B5}] \cdot P[Z_{B5}] + P[Z_{C1}|Z_{B6}] \cdot P[Z_{B6}]$$

where:

probabilities $P[Z_{C1}|Z_{Bi}]$ are read from the horizontal axis in Figure 4-15 and probabilities $P[Z_{B1}]$ through $P[Z_{B6}]$ are shown in Figure 4-12.

$$P[Z_{C1}] = [0.19 \cdot 0.1] + [0.07 \cdot 0.1] + [0.03 \cdot 0.3] + [0.01 \cdot 0.3] + [0.008 \cdot 0.1] + [0.005 \cdot 0.1]$$

$$P[Z_{C1}] = 0.04$$

This computation is repeated to compute P[Z_c] for a range of stages on the interior stage-probability curve as shown in Figure 4-15.

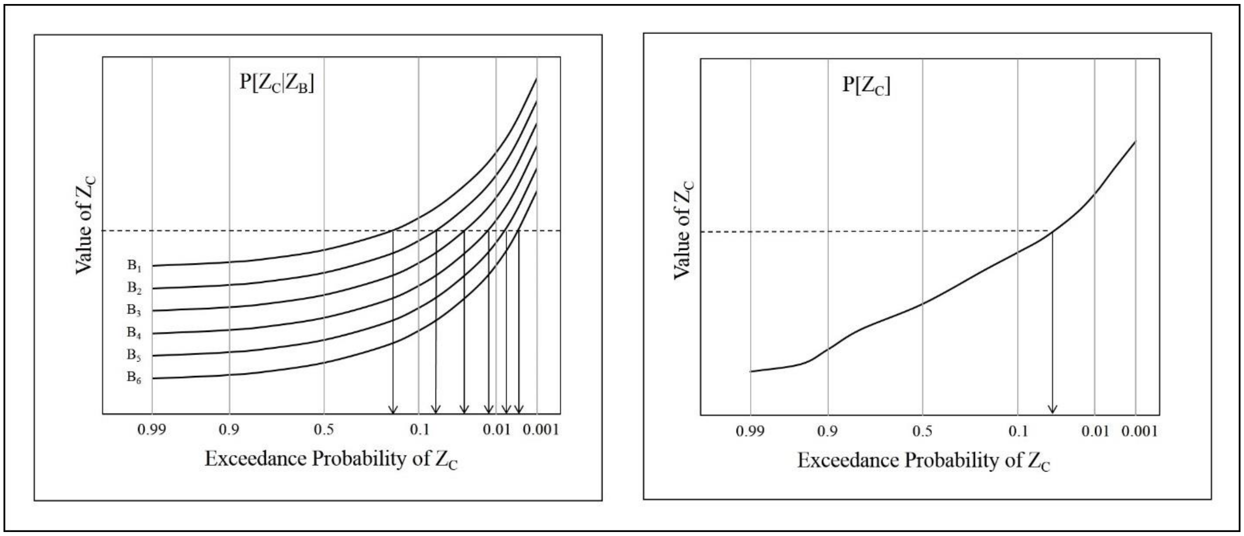

Figure 4-15. Use total probability equation to compute the interior stage-probability curve.

d. Interior runoff and exterior stage are not independent. The above analysis steps are applicable when it is appropriate to assume that the interior runoff and the coincident exterior stage are independent (great interior runoff is coincident with both high and low exterior stages, and no pattern is discernable). However, interior runoff and exterior stage would not be independent when the historic record shows high interior runoff that is coincident with high exterior stages. As discussed in Section 4-2b, the correlation coefficient, R, can be used to determine the degree of correlation between these two variables. Once R is computed, Table 4-4 could be used to assign one of three general degrees of correlation between interior runoff and the coincident exterior stage. In the case of weak correlation, the coincident frequency analysis procedure outlined above can be followed for computing the interior stage-probability curve. Coincident frequency analysis is not required in the case of strong correlation. In this case, the interior pond stage is computed using flows from the interior watershed flow-probability curve and the correspond stage from the coincident exterior stage-probability curve.

Table 4-4. Degree of correlation for ranges of R.

Range of Correlation Coefficient, *R*	**Degree of Correlation**
$0.7 \le R \le 1.0$	Strong
$0.4 \le R < 0.7$	Moderate
$0.0 \le R < 0.4$	Weak

(1) Interior runoff and exterior stage cannot be assumed independent when there is moderate correlation between these two variables. Instead of using one frequency curve for the interior

runoff, conditional-frequency curves for interior runoff given multiple exterior stages are required. Therefore, the coincident frequency analysis outlined above must be modified. Steps 2 through 8 for a coincident frequency analysis where interior runoff and exterior stage are moderately correlated are described below. Step 1 is the same as described above, a stage-duration (or flow-duration) function is developed for exterior stages at primary outlet locations.

(2) Define the conditional runoff-frequency curves for the interior watershed given multiple exterior stages, thereby permitting estimation of $P[Q_A|Z_B]$ for discharge Q_A (an example is shown in Figure 4-16). The exterior stages (or ranges of exterior stage) are provided from the discretized exterior stage duration curve developed in Step 1. For example, the conditional interior runoff-frequency curve for an exterior stage index of B_1 is developed using the annual maximum peak runoff from the interior watershed when the exterior stage is within the range of the discrete segment centered on B_1 from the duration curve. A method for developing the conditional-frequency curves is discussed after Step 8 below. This step should be completed for all relevant conditions, including both without-project and with-project conditions (if proposed measures alter in any way the probability function) and current and future watershed conditions (if the most likely future condition will alter watershed runoff in any manner).

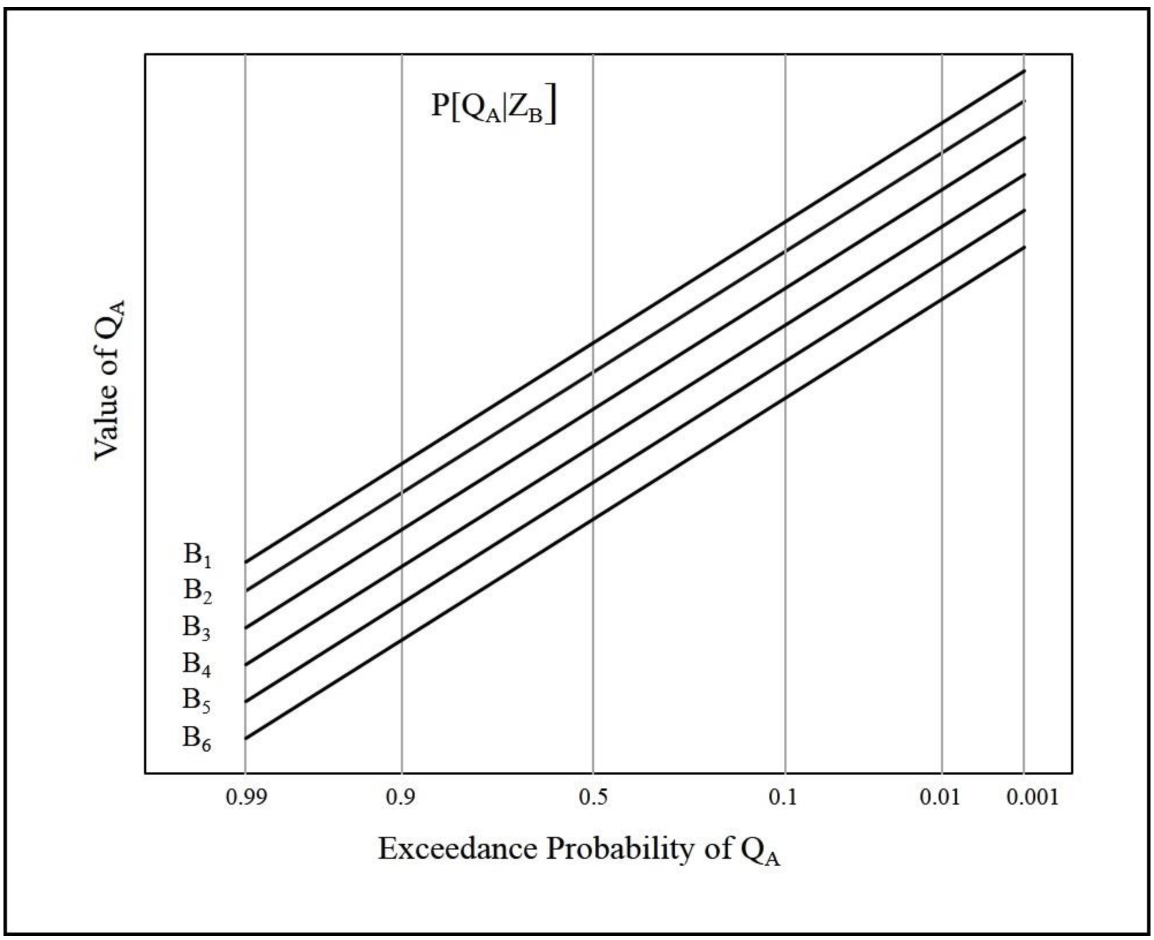

Figure 4-16. Conditional probability curves of interior runoff given multiple exterior stages.

(3) Configure and calibrate a model of the interior facilities. This model will be used to compute interior stage given an interior runoff and an exterior stage.

(4) Select an exterior stage from the discretized duration function from Step 1.

(5) Select a discharge quantile $Q_A|Z_B$ (or a corresponding hydrograph with the same probability) from the corresponding conditional interior watershed runoff-frequency curves, and identify the annual exceedance probability $P[Q_A|Z_B]$. For example, if the exterior stage index B_1 is selected in Step 4, then choose a discharge quantile from the $Q_A|Z_{B1}$ frequency curve. Use this quantile as the upstream boundary condition for the facilities model, with the exterior stage, to compute the interior stage. The probability of this stage, which is a conditional stage is the same as the probability of the interior runoff, $P[Q_A|Z_B]$.

(6) Repeat Step 5 for the entire range of discharge values from the corresponding conditional interior watershed runoff-frequency curve, keeping the exterior stage fixed. The resulting values of Z_C define the conditional interior stage-probability function for the specified exterior stage, $P[Z_C|Z_{Bi}]$.

(7) Return to Step 4 and select another exterior stage value from the discretized duration function and repeat Steps 5 and 6 to define another conditional interior stage-probability function. Make sure to select discharge quantiles $Q_A|Z_B$ from the conditional interior runoff-frequency curve corresponding to the selected exterior stage. Repeat this for all index values defined for the exterior stage duration function.

(8) The computation of the interior stage given interior runoff and exterior stage will produce an array of conditional probability curves for interior stage as shown in Figure 4-14. The computation of the interior stage-probability curve is the same as the case when interior runoff and exterior stage can be assumed independent (discussed in paragraph 4-7c(8)).

(a) Defining the conditional distribution of interior runoff given multiple exterior stages, Step 2, requires a significant amount of data. At least thirty to fifty years of data were required to define an interior runoff-frequency curve. Therefore, ten to 100 times this amount of data was needed to define the conditional interior runoff-frequency curves. Two possible methods for generating this data include: 1) hydrologic modeling by generating climate data using climate models and 2) use Monte Carlo sampling to generate a large sample of annual maximum interior runoff and the coincident exterior stage. The Monte Carlo sampling requires analytical frequency curves (i.e., Log Pearson III, Normal, Log Normal, etc.) for interior runoff and the coincident exterior stage and an estimate of the correlation between these two variables. Using this information, sampled pairs (10,000 or more) of interior runoff and exterior stage can be generated that maintain their correlation and can be used to create the conditional interior runoff-frequency curves.

e. Software. HEC-SSP contains a coincident frequency analysis tool (Table 4-3). The program requires the user to develop the frequency curve(s) for interior runoff and a duration curve for exterior stage (or flow). The coincident frequency analysis helps the user define the index points that discretize the duration curve. Once this information has been specified, it is up to the user to compute the conditional interior stage probability curves (this could be accomplished using HEC-HMS or HEC-RAS) and enter them into HEC-SSP. Then the software will use the total probability equation to compute the interior stage-probability curve.

4-8. Method Selection.

a. General. There is no requirement regarding the use of one method or another; that decision is up to the analyst. Methods are described herein with the idea that the analyst will select the method or combination of methods that is best for the study. Key considerations are described below.

(1) Study goals and objectives. Analysis of interior hydrology is required for a variety of studies, including reconnaissance level and feasibility level flood risk reduction planning, minimum facility identification to support design, studies to define operation rules for pumps, studies to support levee permitting applications, and studies of operation rule changes for a reservoir upstream on the exterior channel. Studies in each of these categories have different requirements, with varying levels of detail needed to answer the questions posed. For example, studies to define operation rules for pumps may not require detailed analyses of low exterior stage conditions, as those are not important for pumping. On the other hand, feasibility level flood risk reduction planning will require consideration of the entire range of exterior conditions in sufficient detail to evaluate alternative risk reduction measures, in sufficient detail, to develop reliable cost estimates. Thus, the methods used for the studies should be consistent with the level of detail required.

(2) Availability of data. Methods identified herein require significant historical weather and water data, along with information that describes in detail the properties of the interior floodplain, interior channels, and facilities at the line-of-protection. For frequency analysis with continuous record, for example, either a long record of streamflow or a long record of precipitation from which streamflow can be synthesized is needed to simulate the behavior of the interior system. If neither record is available, the method cannot be used reliably. Likewise, the coincident frequency analysis method presented herein requires the analyst to assess the correlation between interior and exterior conditions. If data were not readily available to permit that, application of the total probability method described should proceed with caution, considering the presumption of independence inherent in that.

(3) Complexity of interior area. The properties of the interior area and the facilities at the line-of-protection must influence selection of the method used. For many simple systems (such as the system illustrated in Figure 4-2), either of the methods described in detail herein are satisfactory. However, for more complex systems - those with multiple streams and drainage facilities, for example - application of the total probability method may quickly overwhelm the

capability of well-known statistical analysis tools. For example, if flow into the pond at the line-of-protection consists of runoff from multiple upstream subbasins, the coincidence of those runoff events must also be considered. This leads to the need for a smaller scale coincident frequency study embedded within a larger study. Establishing and accounting for correlation and dependence in that case may be challenging. The alternative - a period of record analysis coupled with frequency function fitting - accounts explicitly for the complex coincidence structure, so may be a simpler choice for this case.

(4) Properties of alternatives to be evaluated for planning study. Hydrologic analysis of an interior area commonly is undertaken to support a flood risk reduction study, and thus the analysis must consider both without-project and with-project conditions within the study area. Accordingly the analysis approach must be designed to permit simulation of the impacts of measures proposed. For example, if an alternative for damage reduction at the line-of-protection is a reservoir or detention storage upstream in the interior watershed, the models used must be capable of simulating the hydrologic impacts of that storage. And to achieve this, discharge hydrographs must be available to represent the inflows to the storage, thereby permitting analysis of the peak reduction possible. In that case, development of the upstream discharge frequency function only, which may be adequate for coincident frequency analysis of without-project condition, will not provide the necessary information. Hydrographs (either historical or synthesized) must be computed for routing. This need must be anticipated as the hydrologic engineering management plan is developed during study design.

(5) Skill of analyst. Even the simplest methods described herein are complex. The analyst must develop and use conceptual models of watershed runoff, open channel flow, flood storage, pumping, and gravity drainage (culvert flow). The analysts must also fit and interpret frequency functions and manipulate and combine those if the total probability method is selected. The level of skill and experience of the analyst must be weighed as a method is selected. If technical assistance is required, the USACE centers of expertise should be consulted.

(6) Resources available. Selection of a method must consider the time and money available for the study, and the analysis methods must be scaled appropriately. A reconnaissance level planning study, for example, requires identification of flood risks and assessment of solutions with limited budget. In that case, configuration, calibration, and application of models for synthesis of a period of record likely are infeasible, and other options must be considered. In that case, it might be possible to develop reasonable estimates of discharge quantiles with a regression equation or another of the procedures described in EM 1110-2-1415 and other USACE guidance on watershed runoff analysis.

(7) Local preference and experience. USACE analysts, USACE cost sharing partners, and local consultants will carry out the study using the method(s) selected. In some cases, local USACE analysts will have completed previous hydrologic analysis of interior areas with preferred methods. Likewise technical analysts of local cost sharing partners or consultants to the USACE or the local partners may have experience with certain methods that are consistent

with those described herein. This local preference and experience should be considered when selecting a method so as to build on the knowledge that is available.

b. Study Strategy. In most cases, analysts will discover that due to the unique nature of the watershed, channels, study needs, resources, and other factors, "off the shelf" solutions will not meet the study needs. In that case, a hybrid or customized study strategy should be designed. For example, a study may require that certain elements of the frequency analysis with continuous record method be used to establish the required interior area discharge frequency function, which then will be used within the context of the coincident frequency analysis framework.

c. USACE Studies. For USACE planning studies, it is common for work to progress in phases or stages. At each stage, additional detail is added and the analysis is fine tuned. Accordingly, the method selected for hydrologic analysis of an interior area should be adaptable to this shifting study strategy, and it should be sustainable so that the method can be used in successive iterations of flood risk reduction plan formulation and evaluation. This means, in general, that the methods used should be consistent generally with methods described herein, that the computations done should use commonly available and well understood software, that data be managed properly, and that study documentation be complete and contemporaneous.

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CHAPTER 5

Analysis Methods and Procedures for Coastal Interior Areas

5-1. General.

a. Overview. Chapter 4 describes analysis of interior areas protected by a levee or floodwall from overflow from an exterior channel, such as a river or stream, driven by hydrologic events (precipitation/runoff). For the case where the exterior area (coastal), such as an ocean, bay, estuary, or large lake, the interior analysis is the same as discussed in Chapter 4, but overflow from the exterior area may include storm surge, waves, and tides. This chapter concentrates on the analysis of the exterior water levels and overtopping of the line-of-protection for analysis of interior areas adjacent to coastal areas. Examples of such coastal areas include:

(1) Southeastern Louisiana where cities are protected by levees and seawalls from hurricane storm surge and waves generated in the Gulf of Mexico, Lake Pontchartrain, and various bays.

(2) Areas in south central Florida that are protected by the Herbert Hoover Dike from hurricane storm surge and waves generated in Lake Okeechobee.

(3) South San Francisco Bay where cities are protected by levees from combined tides and wind generated storm surge and waves.

(4) Manhattan where the Battery seawall protects New York City from tides, storm surge, and waves generated in the Atlantic Ocean and Lower and Upper New York Bay.

5-2. Basic Concepts.

a. Surge and waves. Flooding in coastal systems is typically a result of storm generated surge and waves. Storms are atmospheric disturbances with low atmospheric pressure and strong winds (Scheffner, 2002). The main contributors to increased water levels during a coastal storm are the wind stress, wave stress, and low atmospheric pressure at the storm center. The largest contributor to storm surge is typically the wind blowing over the water. The magnitude of the wind-driven surge is a function of the wind speed (higher surge with stronger winds), wind direction, water depth (higher surge for shallower basins), and the distance over which the wind blows. The wind-driven vertical increase in water level can be a few feet to a few tens of feet. The wave stress component of storm surge is called wave setup. Wave setup results from the transfer of momentum from breaking waves to the water column and can result in increases in water level of approximately fifteen to twenty percent of the breaking wave height. Wave setup can contribute several feet to the total storm surge in areas directly exposed to waves. The atmospheric pressure component of storm surge is called the barometric tide. This component is a bulge of water due to the lower atmospheric pressure in the center of the storm. The barometric tide is a few feet or less. Storm surge increases the water level for periods of several hours to days. Storms are classified as tropical storms or extratropical storms. Tropical storms have warm cores and originate in the tropics. Extreme tropical storms are called tropical cyclones, hurricanes (Atlantic, Gulf of Mexico, and eastern Pacific), or typhoons (western Pacific). Hurricane Katrina is an example of an intense hurricane that produced a storm surge of almost thirty feet on the Mississippi Coast. Hurricanes tend to produce the most extreme water levels in the southeast Atlantic and the Gulf of Mexico. The duration of a tropical storm affecting a coastal area tends to be relatively short (less than one day) and the coastal area impacted is generally relatively small (less than fifty miles). Storms that result from the interaction of warm and cold fronts are called extratropical storms. These cold core storms generally produce the largest storm surge along the United States Pacific Coast and North Atlantic Coast. Extratropical storms can be long in duration (several days) and impact large regions of the coasts (several hundred miles).

b. Tides. Astronomical tides are the periodic rising and falling of Earth's oceans in response to the gravitational attraction of the Moon, Sun, and all other celestial bodies and the rotation of the Earth. The Moon has the strongest effect on the tides because it is nearest the earth. The timing and elevation of the tides varies from location to location and are influenced by the relative alignment of the Sun and Moon, the shape of the coastline/basin, and the near coast water depths. High tides may occur approximately twice a day (semidiurnal with a period of approximately 12 hours 25 minutes), once a day (diurnal), or a mixture of the two (mixed tide). The tidal range (the difference in height between high and low water) varies on a two-week cycle, with the maximum range (spring tide) occurring when the Moon and Sun are aligned relative to the Earth (at the time of a new or full moon) and the minimum range (neap tide) occurring when Moon and Sun are separated by 90 degrees when viewed from the Earth. Smaller amplitude variations occur in a nineteen-year cycle. Tides alone are rarely a source of flooding, but high tides in addition to storm surge and wave setup can impact flooding. For example, a hurricane making landfall on high tide can result in significantly higher total water levels than the same storm landfalling at low tide.

(1) Storms and tides are independent (uncorrelated) in the analysis of water levels, but storm surge and waves are generally dependent (generated by the same storm event and peaking at approximately the same time). The water levels in the interior regions may or may not be correlated with coastal storms. For example, rain fall and runoff associated with a slow moving hurricane or extratropical storm may be correlated to the storms surge. But, flooding from a large river within the interior basin would likely be independent of coastal storms. An example of independence is that most hurricanes occur in August-October when river flow is generally low, thus these exterior/interior flooding events are physically independent, but statistically (negatively) correlated.

5-3. Procedure Overview.

a. General. The hydrologic procedures for analyzing water levels with and without coastal interior projects are similar to those described in Section 4.3 for riverine interior areas. Significant differences are as follows:

(1) Coastal exterior flooding results from high intensity, short duration storms. These storms impact the coast for hours to days, whereas riverine events may be slow events that develop through combined snowmelt and rainfall/runoff throughout a large basin and last for days to weeks.

(2) Coastal storms in some regions include two populations: extratropical and tropical storms. The hazard analysis technically will depend on the dominant type of storm. Extratropical and tropical storm populations have different characteristics and should not be mixed.

(3) Because landfalling hurricanes generally impact a relatively small stretch of coast, hurricane events at a given project site are relatively rare. Thus, the historical record of water levels and storm parameters may not accurately represent the frequency of occurrence of hurricane flooding (too few measured events over too short of a record). Modeling of hypothetical storms is used to supplement the analysis. NOTE: This chapter focuses on determination of the analysis of the exterior (coastal) stage-probability curves to be applied in a coincident frequency analysis.

5-4. Data Requirements.

a. Data Types. In addition to the hydrologic data requirements for the interior basin, discussed in Section 4-4, the following data were required for the exterior water level analysis: historical exterior water level, wave and wind data; historical storm parameters; storm wind fields; tide data; river inflow; bathymetry, topography, and land use classifications for the exterior basin; operational procedures for closure structures; and projections of subsidence and sea level rise.

(1) Historical exterior water level, wave, and wind measurements. If a sufficiently long, continuous record of water level is available at the site(s) of interest, the data can be used directly to develop the exterior stage-probability curve. A forty to fifty year record is required to establish the water level with a one-percent probability of occurrence in a year. Water levels are measured at many locations along the United States coastline, and the National Oceanic and Atmospheric Administration's (NOAA) National Ocean Survey (NOS) has archives of measurements accessible through the Tides and Currents web page (http://tidesandcurrents.noaa.gov/index.shtml). But, it is rare that sufficient data is available. Typically, measurement stations are not at the location(s) of interest, data gaps exist during storms, and/or the period of record is too short. Wind measurements can be used to establish the extratropical storm climate. Wind, wave, and water level measurements are also valuable to identify coastal storm events and verify models for computation of surge and waves.

(2) Historical storm parameters. Tropical and extratropical storms are parameterized in different ways. Hurricanes can be defined by a small set of storm parameters: central pressure, radius of maximum winds, forward speed, track (including landfall location and angle relative to the coast), and the Holland B asymmetry parameter (Holland, 1980). Historical hurricane

parameters are available from the National Hurricane Center for the Atlantic basin (including the Gulf of Mexico) and the Central North Pacific (Hawaii) (http://www.nhc.noaa.gov/pastall.shtml#climo). Considering only data from 1940 and more recent because of observational limitations in the earlier data is recommended. Probabilities of each parameter are combined using a Joint Probability Method (JPM) to determine the hurricane hazard. Extratropical storms are not easy to parameterize. But, extratropical storms occur more frequently than hurricanes and impact longer stretches of coast line (e.g., several northeaster impact large stretches of the East Coast each year). The USACE Wave Information Studies (WIS) (http://wis.usace.army.mil/) database provides a consistent set of simulated winds and waves for U.S. coastlines (twenty to forty year record) for identifying storm climate. The WIS wind and wave fields can also be used as boundary conditions for running surge and near shore wave models.

(3) Storm Wind Fields. Once a suite of storms is identified through analysis of the storm climate, wind fields can be developed through parametric models (e.g., Planetary Boundary Layer (PBL) model for hurricanes) or from historic data and kinematic analysis (e.g., WIS wind fields). The wind fields are then used to drive numerical models of storm surge and waves as input to JPM or EST (Empirical Simulation Technique) methods to estimate stage-probability curves.

(4) Tide Data. Tides are included in the measurements and archives maintained by NOS. NOS also provides tide predictions at tidal stations throughout the United States. These data can be used to determine mean, spring, and neap tidal ranges (http://tidesandcurrents.noaa.gov/index.shtml).

(5) River Flow and Stage Data. When a river discharges into the coastal area of interest, storm surge may propagate up the river and impact exterior water levels or induce overtopping on river levees or structures (e.g., the Mississippi River in New Orleans). The river discharge and stage impacts the speed at which the surge travels up the river and the base water level the surge builds on. If flooding from the river side is significant, river flow may be used as another parameter in the JPM analysis. If river flow is less critical, monthly average flows may be used.

(6) Bathymetry, Topography, and Land Use Classifications for the Exterior Basin. The requirement for topography in the interior areas was already discussed in Session 4-4. Bathymetry and topography are also required in the exterior basin for either numerical or analytical modeling of storm surge and waves. Both storm surge and waves are sensitive to water depth, bottom slope, and basin geometry. Land Use Classification is required to establish bottom roughness in surge and wave models and to establish sheltering of the wind by tree canopies.

(7) Operational Procedures for Closure Structures. Flood gates and other closure structures would be closed in major storm events. Closure of gates generally has a negligible effect of exterior water levels, but the timing of the closure can significantly impact interior water levels due to pre-storm water build ups on either side of the structure.

(8) Projections of Subsidence and Sea Level Rise. One of the significant uncertainties in coastal design is the magnitude of sea level rise and subsidence. Global estimates of the current rate of sea level rise are approximately 1.7 millimeters/year, but future projections vary greatly. Also, the current local relative rate of sea level rise (including subsidence) may be many times the global rate (e.g., in Louisiana, rates of 5.7 to 12.7 millimeters /year have been estimated. Guidance on incorporating sea level rise in project design is provided in ER 1100-2-8162. Relative sea level rise in the past was considered as only a linear addition to the water levels at levees and sea walls, but modeling studies have shown nonlinear increases in surge and wave heights, particularly in areas fronted by shallow wetlands (Smith, 2010).

5-5. Hurricane (Tropical Storm) Analysis.

a. Several different methods have been applied in the past to estimate extreme hurricane water levels and waves (Standard Project Hurricane, estimates based only on historical storms, JPM, EST, and Empirical Track Model). Following the forensic study of Hurricane Katrina by the Interagency Performance Task Force (IPET, 2006), an improved, risk-based JPM approach was developed to estimate hurricane water level probabilities (Resio, 2007; USACE, 2007). The study team will need to evaluate available information, observed data, and budget when selecting a modeling method.

b. In the JPM approach, a limited number of hurricane parameters are selected to cover the range of hurricanes potentially impacting the area of interest. These parameters are used to drive a series of coupled models for winds, waves and surge to determine the peak water level at the point of interest for each storm and estimate overtopping of the protection measures, if any. A probabilistic model is then used to estimate the surge elevation, wave height, and wave period frequency curves at the locations of interest. Additionally, the probabilistic model provides error estimates associated with the water level, wave, and overtopping estimates.

c. As in previous studies, hurricanes are defined by five storm parameters:

(1) c_p is central pressure

(2) R_p is the scaling radius of the pressure field (similar to the radius of maximum winds)

(3) v_f is the forward velocity of the storm

(4) θ_l is the track angle relative to the coast at landfall

(5) x is the distance between the point of interest and the landfall location

d. In past studies, all parameters were assumed to remain constant as a hurricane approached the coast and made landfall. Trends show that storms tend to fill by about ten to fifteen millibars (mb) and become slightly larger (fifteen to thirty percent) over the last 90 nautical miles (NM) of coastal water before landfall (Resio, 2007). Hurricane parameters are defined at landfall, but the c_p and R_p are transformed to be representative of offshore values and varied through the last 90 NM before landfall. The Holland B parameter describes the peakedness of the hurricane wind fields. Resio (2007) found that the Holland B parameter in mature storms within the Gulf of Mexico fell in a narrow range and thus adopted a constant value of 1.27 for storms centered more than 90 nautical miles (NM) from the coast and decreases to a value around 1.0 (less peaked) at landfall. An additional improvement, is representing hurricane tracks as curved instead of straight lines, based tendencies shown in historical tracks (e.g., in the Gulf of Mexico, the strongest storms tend to enter the Gulf through either the gap between Cuba and the Yucatan Peninsula or through the southern Florida to Cuba area).

(1) Selecting the values of hurricane parameter for simulation requires examination of historical hurricane data (e.g., http://www.nhc.noaa.gov/pastall.shtml#climo), considering only data from 1940 and later. The first consideration is the series of hurricane tracks that will be simulated. The track is the time history of hurricane position through the basin. The track takes into account the angle relative to the coast at landfall and the landfall location (θ_l and x). An example of some of the tracks applied in southeastern Louisiana is shown in Figure 5-1. Typically, tracks are shifted along the coast to give alongshore spacing at landfall on the order of the pressure radius of the hurricane (tracks in the southeastern Louisiana study were spaced 31 NM). The central pressure and pressure radius determine the intensity and size of the storm. The forward speed of the storm impacts the maximum wind speed (which increases with forward

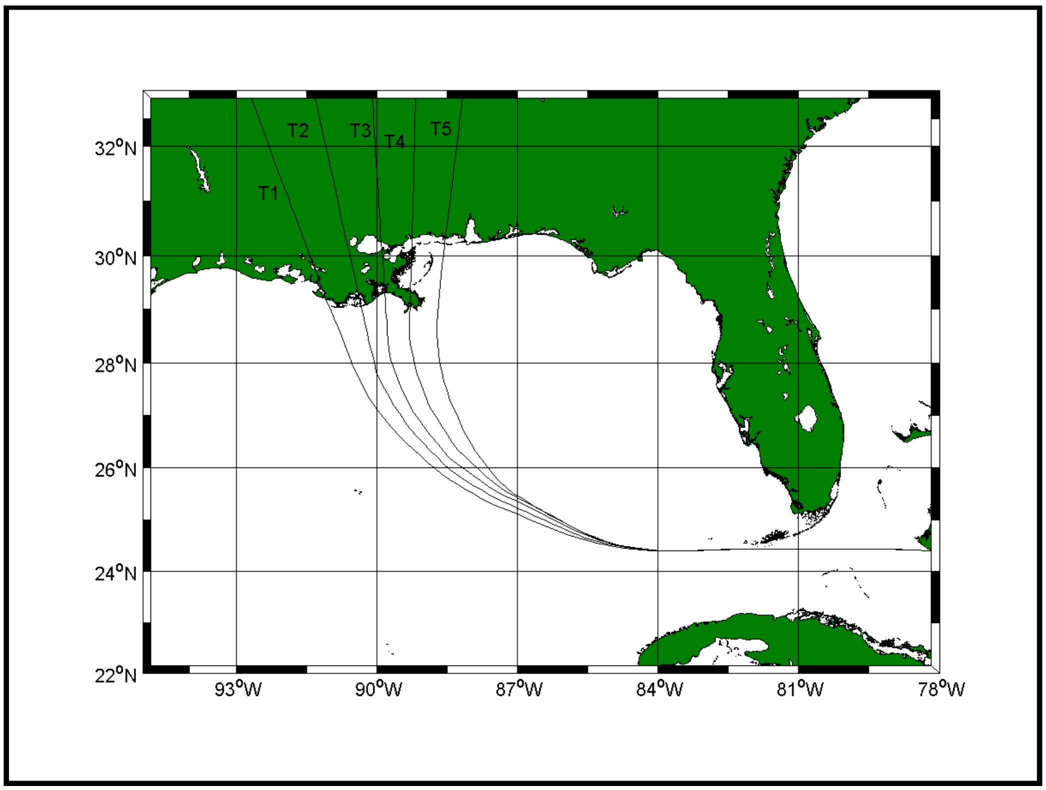

Figure 5-1. Sample of tracks used for JPM-OS analysis for SE Louisiana.

speed) and the maximum surge and waves (which may increase with slower forward speed due to the increased duration of the forcing). The selected parameters should cover the range observed within the region of interest. The selection can be optimized by eliminating combinations of parameters that do not produce significant surge (e.g., small, low intensity storms, very oblique landfalls) and by coarsening the resolution of the parameters suite when the response is linear (e.g., surge is linearly related to central pressure near the landfall location). Numerical simulations may be used to the estimate the sensitivity of the response (surge and waves) to the hurricane parameters for a given study site. Resio (2007) discusses the optimal sampling method used in the Join Probability Method with Optimal Sampling (JPM-OS) anlaysis for southeastern Louisiana in detail.

(2) Once the hurricane parameter suite for simulation has been selected, the next step is numerical simulation of the winds, surge, and waves. Figure 5-2 shows the progression and linkages of the modeling procedure.

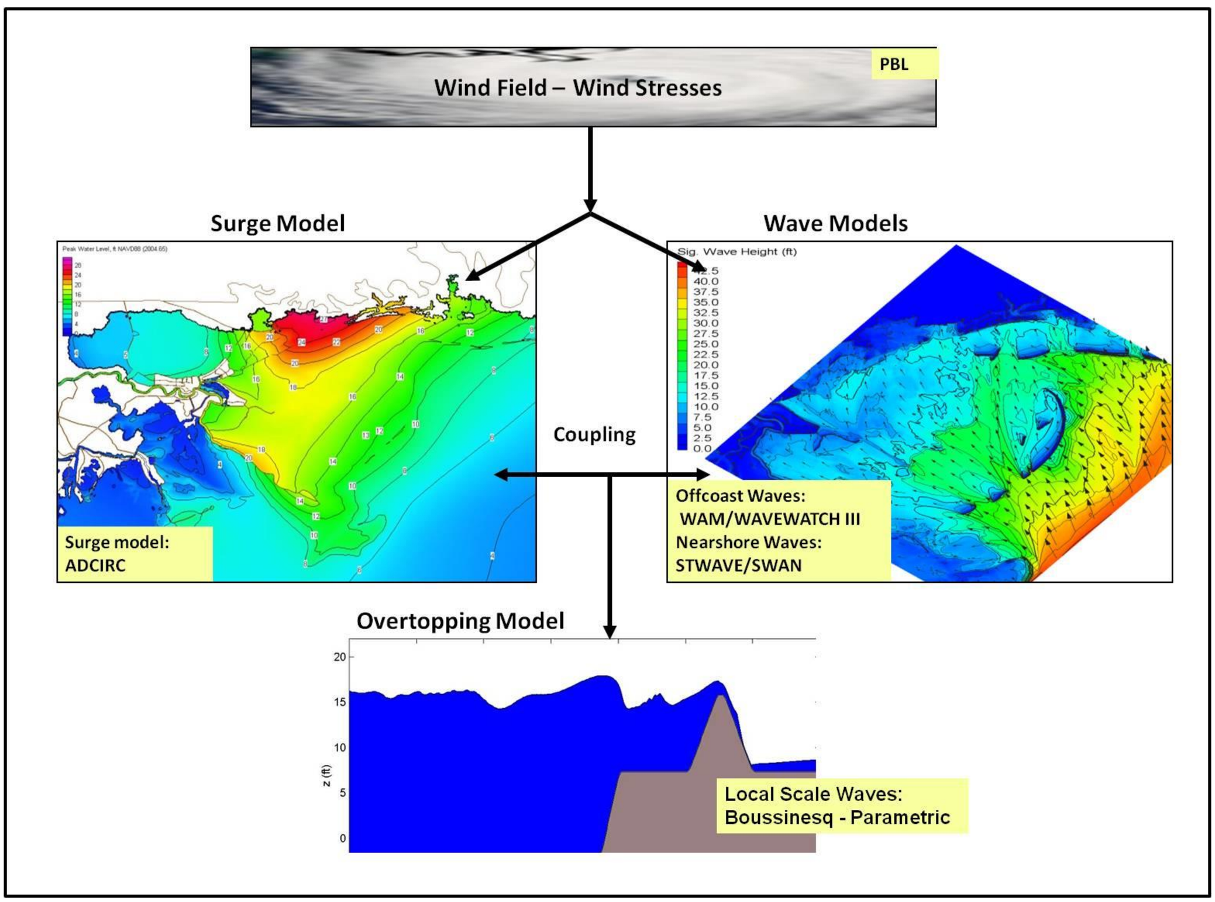

Figure 5-2. Modeling procedure.

(a) Winds and Pressure. Winds and pressures are modeled with a marine PBL model that links the marine wind profile to large scale pressure gradients and thermal properties. The

Tropical Cyclone 96 Model (Thompson, 1996) is used to generated dynamic hurricane wind and pressure fields based on the storm parameters c_p, R_p, v_f, Holland B, and hurricane track.

(b) Surge. Wind-, pressure-, and wave-driven surge are modeled with a circulation model based on conservation of continuity and momentum. The ADvanced CIRCulation Model (ADCIRC) (Luettich, 2004) solves the generalized wave-continuity equation on linear triangular elements and is used to generate the dynamic water levels. The finite element formulation allows high resolution in areas of interest (approximately thirty meter resolution) and coarse resolution away from the coast (up to five kilometer resolution). ADCIRC is tightly coupled to a near shore wave model, which provides the wave stresses. In addition to the wind, pressure, and wave stress fields, ADCIRC requires input of bathymetry/topography and bottom roughness (to represent frictional losses). Bottom rough coefficients are derived from land use maps.

(c) Waves. Wind-driven waves are modeled in two steps. First the waves are modeled on a large scale that represents the generation and propagation of waves along the track of the hurricane (e.g., the entire Gulf of Mexico) at a resolution of approximately five to ten kilometeres. The Wave Prediction Model (WAM) (Komen, 1994; Gunther, 2005) is used to generate the offshore waves and provide boundary conditions to near shore wave modeling. The generation model inputs include the hurricane wind fields and the basin geometry and bathymetry. An alternative wave generation/propagation model is WAVEWATCH III (Tolman, 2009). At water depths where the waves begin to interact strongly with the sea bottom, wave information from the generation-scale model are applied as boundary conditions for a near shore wave generation/transformation model (depths approximately thirty to fifty meters) that resolves the bathymetry features of interest (approximately 200 meter resolution). The STeady-state WAVE model (STWAVE) is used to transform waves into the coast, including areas flooded by surge. The near shore model includes the processes of wave generation, refraction, shoaling, and breaking. The transformation model input include hurricane wind fields, bathymetry/topography, boundary conditions from the wave generation model, water levels from the surge model, and bottom roughness (to represent frictional losses). An alternative wave transformation model is SWAN (Simulating Waves Nearshore) (Booij, 1999; Zijlema 2010). If the foreshore in front of the structure (levee or seawall) is shallow (ratio between the significant wave height, H_s, and the water depth, d, is small ($H_s/d > 1/3$) and the foreshore length (L) is longer than one deep water wave length L_o, or $L > L_o$ where $L_o = gT_p^2/(2\pi)$), the wave height given by the model should be limited to give to a maximum of $H_s = 0.4d$ at the structure toe.

(d) Overtopping. In the case that the line-of-protection is overtopped, an overtopping model must also be applied. The overtopping may have two components: the weir-like flow due to the still water level exceeding the top of the line-of-protection, and the intermittent overtopping due to waves. The levee or seawall toe is in relatively shallow water, even during extreme surge events, and this means that the waves become very nonlinear (significant low frequency energy and peaked wave crests). Accurate modeling of wave overtopping requires high-resolution (approximately one meter), computationally intensive models. Three approaches may be applied.

- The first approach is application of a parametric relationship that is based on laboratory studies of wave overtopping in a flume. For straight and bermed impermeable slopes (e.g., levees), the overtopping formula by Van der Meer (1995) provides an estimate of the overtopping discharge per unit crest width. The input parameters are wave height at the structure toe, wave period, slope of the levee, and freeboard (water level). Additional coefficients are included to account for the surface roughness of the slope, the influence of a berm (if any), the influence of shallow depths on the wave height distribution, and the long- or short-crestedness of the waves (actively generated hurricane are classified as short crested). For vertical walls, the overtopping formula by Franco (1999) provides an estimate of the overtopping discharge per unit crest width. The input parameters are wave height at the structure toe, freeboard, wall front geometry, and long- or short-crestedness of the waves. EM 1110-2-1100 and Burcharth (2011) provide details on application of the overtopping formulas for levees (EM 1110-1-1110, Part VI, Table VI-5-11, page VI-5-32) and for walls (EM 1110-1-1110, Part VI, Table VI-5-13, page VI-5-34). Similar equations are given by Van der Meer (2002) and USACE (2007).

- The second approach is application of a process-based numerical model. In this approach the overtopping discharge is computed using the equations for mass and momentum of fluid motion. A Boussinesq model is presently the most appropriate model to compute these parameters within a reasonable time frame (e.g., the COULWAVE (Cornell University Long and Intermediate Wave) model (Lynett, 2002; Lynett, 2004). The inputs to the Boussinesq model are the wave parameters at the levee toe, the water level, and the bathymetry/topography (levee geometry). Boussinesq models have the advantage of being able to handle complex geometries that are not covered by the parametric approach, but the application is very time-consuming. The Boussinesq model can be run for one-dimensional, representative levee cross sections to reduce run times. The Boussinesq model cannot be used for vertical walls.

- The third approach is estimating overtopping discharge through physical modeling. This approach does not limit the complexity of the structure or limit the incident wave conditions, but is generally an expensive alternative.

(e) If a levee is overtopped, there is a potential for failure due to erosion. Levee degradation depends on velocity at the crest and unprotected side, soil type, vegetation cover, levee construction, and maintenance. The allowable overtopping rates before concerns of breaching are 0.1 cfs/feet for clay covering and a grass cover according to the requirements for the outer slope or for an armored inner slope, grass covered earthen levees, 0.01 cfs/feet for clayey soil with a reasonably good grass cover, and 0.001 cfs/ft for sandy soil with poor grass cover (USACE, 2007). The allowable overtopping rate for floodwalls with appropriate back side protection is 0.03 cfs/feet.

5-6. Cumulative Density Function. The cumulative density function (CDF) for surge is defined as (Resio 2007):

$$F(\eta) = \int\int\int\int\int\int p(c_p, R_p, v_f, \theta_l, x) p(\varepsilon|\eta) H[\eta - \Psi(c_p, R_p, v_f, \theta_l, x) + \varepsilon] dc_p\, dR_p\, dv_f\, d\theta_l\, dx\, d\varepsilon \quad (5\text{-}1)$$

where:

- η is the surge level of interest,
- $p(.)$ represents probability,
- $H[.]$ is the Heavyside function,
- Ψ is the maximum modeled surge level for given model input parameters x_1 to x_n, and
- ε is the error

a. The error term, ε, is due to tides, wind field deficiencies, model deficiencies, and unresolved scales. The probabilities associated with the hurricane parameters are estimated based on historical data available through NOAA. Only since the 1940s have aerial reconnaissance, radar, and other sensing technologies enabled hurricanes to be characterized accurately (USACE, 2010). Thus, it is recommended to use hurricane data for 1940 and later, unless a major hurricane has impacted the region prior to the 1940s and can be reliably characterized. A detailed analysis of hurricane probability for the New Orleans area is given by Resio (2007). In this analysis, the spatial variability and interrelationship of the hurricane parameters is developed based on twenty-two hurricanes with a c_p of 955 mb or less. The probability of a given set of hurricane parameters is a function of the probability of, c_p (parameterized by a Gumbel distribution), given that a hurricane occurred, the probability of a certain, R_p, as a function of c_p, the probability of, θ_l, given the landfall location (x), the probability of, v_f, given θ_l, and the frequency of storms per year given the landfall location. Parameter relationships were developed for on degree segments of the coastline using a hurricane sample within plus or minus three degrees along the coast.

b. The error term, ε, has four components that are assumed to be independent:

(1) Tides

(2) Random variations in Holland B and other asymmetries not represented by the PBL

(3) Track variations not captured in storm set

(4) Model errors (including errors in bathymetry, model physics, roughness coefficients, wind field variations from PBL, etc.)

c. The tide component can be estimated by linear superposition and represents the percentage of time occupied by a given tidal stage, which can be derived from tidal data. The surge varies approximately linearly with the Holland B parameter. The track variations mainly affect the wave field and thus wave setup contributions to storm surge. A reasonable approach is to assume the surge deviations due to track variations are Gaussian with a standard deviation related to the wave setup contribution to surge (for Louisiana work, a standard deviation of twenty percent of the wave setup contribution was assumed (Resio, 2007)). Model errors can be estimated from verification (using gage or high-water mark data) and sensitivity simulations.

Resio (2007) estimated standard deviations of 2.0 to 3.5 feet for the combined wind/surge/wind modeling for New Orleans. The probability of a given error ε is given by:

$$p(\varepsilon) = \iint\iint \delta(\varepsilon_1 + \varepsilon_2 + \varepsilon_3 + \varepsilon_4 - \varepsilon)\, p(\varepsilon_1)\, p(\varepsilon_2)\, p(\varepsilon_3)\, p(\varepsilon_4) d\varepsilon_1 d\varepsilon_2 d\varepsilon_3 d\varepsilon_4 \quad (5\text{-}2)$$

where $\varepsilon_1, \varepsilon_2, \varepsilon_3,$ and ε_4 are the independent errors due to tide, Holland B parameter, track variation, and model error. The Dirac delta function selects the combinations of errors that result in a total error of ε.

The mean CDF for wave height and associated peak period are calculated neglecting errors, where y represents either wave height (H_{mo}) or period(T_p):

$$F(y) = \int\int\int\int\int\int p(c_p, R_p, v_f, \theta_l, x) H[y - \Psi(c_p, R_p, v_f, \theta_l, x)] dc_p\, dR_p\, dv_f\, d\theta_l\, dx\, d\varepsilon \quad (5\text{-}3)$$

The errors around these mean values are represented by a normal distribution with a standard deviation of ten percent for wave height and twenty percent for wave period, based on validation studies.

The CDF for overtopping discharge rate is calculated with a Monte Carlo simulation, assuming peaks in water level and wave height occur at the same time. The Monte Carlo analysis is executed as follows:

(1) Realization of water level. A random number between zero and one is selected to set the exceedance probability, p, for water level. The appropriate mean water elevation is selected from the CDF (Equation 5-1) for the given return period (e.g., $\overline{\eta_{1\%}}$). Assuming a normal distribution and standard deviation estimate for the water level, σ, the water level exceedance probability is applied to estimate the realization of the water level η,

$$\text{p} = 0.5\left[1 + erf\left(\frac{x - \bar{\mu}}{\sqrt{2\sigma}}\right)\right] \quad (5\text{-}4)$$

where:

x is the parameter of interest (η in this case),
$\bar{\mu}$ is the mean value of the parameter ($\overline{\eta_{1\%}}$ in this case), and
erf is the standard error function (see math texts for numerical solutions). NOTE: This intrinsic function can be solved iteratively for, η, or a lookup table can be used (see statistics text books).

(2) Realization of wave height and period. A random number between zero and one1 is selected to set the exceedance probability, p, for the wave height. The appropriate mean wave height from the CDF (Equation 5-3) for the given return period (e.g., $\overline{H_{mo1\%}}$). Assuming a normal distribution and standard deviation estimate for the water level, σ, the wave height exceedance probability is applied to estimate the realization of the wave height using Equation

5-4. Repeat procedure for wave period.

(3) Realization of overtopping rate. If overtopping is calculated with the empirical formulas in EM 1110-2-1100, then, a random number between zero and one is selected to set the exceedance probability, *p*, for the coefficient in the equation. Assuming a normal distribution and the mean and standard deviation of the coefficient given in EM 1110-2-1100, the coefficient exceedance probability is applied to estimate the realization of the coefficient using Equation 5-4. Apply this coefficient and the water level, wave height, and period calculated in Steps 1 and 2 in EM 1110-2-1100 to calculate the realization of overtopping rate. If a numerical or physical model is used to estimate overtopping rates, overtopping is estimated from the water level, wave height, and wave periods (in the form of a look up table), and the rate is randomized using the standard deviation of overtopping from the simulations. Repeat Steps 1 through 3 a large number of times (N > 1,000 – 10,000).

(a) A mean value for the overtopping rate can then be calculated from the N realizations. Confidence limits can also be estimated. Guidelines for establishing the overtopping rate threshold (onset of levee erosion and damage) for different types of embankments can be found in EM 1110-2-1100 Table VI-5-6 (Van der Meer, 2002). The following wave overtopping rates have been established for the New Orleans District hurricane protection systems (USACE, 2007):

(b) For the one percent exceedance still water, wave height and wave period, the maximum allowable average wave overtopping of 0.1 cfs/feet at 90 percent level of assurance and 0.01 cfs/feet at 50 percent level of assurance for grass-covered levees;

(c) For the one percent exceedance still water, wave height and wave period, the maximum allowable-average wave overtopping of 0.1 cfs/feet at 90 percent level of assurance and 0.03 cfs/feet at 50 percent level of assurance for floodwalls with appropriate protection on the back side.

CHAPTER 6

Flood Risk Management Measures

6-1. Overview.

a. General. This chapter describes measures that may be deployed to reduce flood risk in an interior area; a floodplain area that is protected by levees or floodwalls (line-of-protection). These measures include the levees or floodwalls that are provided to reduce the risk of flood from the exterior river, lake, or sea. The measures described herein also include those necessary to reduce the risk of flooding due to excess precipitation in the interior area. Runoff from such precipitation can no longer flow into the exterior channel, and thus creates a hazard unless properly managed. The measures identified should be considered in planning investigations, combining them as appropriate to formulate alternatives that will be considered in a nominate-simulate-evaluate-iterate plan selection strategy.

b. Measures. Measures identified herein are categorized as structural if they alter the nature of the hazard to flooding or nonstructural, if they alter the exposure or consequence of current or future flooding. Measures described can be implemented or remote from the line-of-protection.

c. Detail. The level of detail for analysis of measures and alternatives must be consistent with the purpose of the study. Early in a planning study, the level of detail must be sufficient only to establish a federal interest in the project. As the study moves forward, detail must be sufficient to refine identification of the NED (Net Economic Development) plan to the point that significant changes in cost, engineering performance (Annual Exceedance Probability (AEP), Conditional Non-Exceedance Probability (CNP or Assurance), etc.), or benefits will be avoided in the design phase of the study. As the planning progresses to design, additional detail is required to define dimensions, operation policies, and so on.

6-2. Structural Measures at the Line-of-Protection.

a. Structural Measures. Structural measures commonly deployed at the line-of-protection include:

(1) Line-of-protection (levees and floodwalls)

(2) Gravity outlets

(3) Detention basins

(4) Pump stations

(5) Intercepting sewers or channels

(6) Pressure conduits

(7) Seepage and relief wells

b. Line-of-Protection. The line-of-protection comprises the main levees or floodwalls. The objective of the line-of-protection is to reduce the hazard to and therefore lessen the risk of direct flooding of the interior area due to elevated water levels in rivers, lakes, or seas on the exterior side of the line-of-protection.

(1) While blocking flow onto the floodplain adjacent to a river, lake, or sea, the levees or floodwalls also intercept and block runoff from the protected interior floodplain. Creeks, streams, and natural drainage from the floodplain can no longer convey water into the exterior river, lake, or sea. That water ponds and potentially inundates property and threatens life safety in the area protected from exterior flooding. This exchanges one source of flood risk for another, albeit a lesser risk in most cases. The interior water must be managed to minimize the risk.

(2) The line-of-protection should be aligned to minimize the size of the interior area for which additional drainage must be provided and to preserve, to the extent possible, the natural conveyance of floodplain waters into the exterior lake, river, or sea. This can be accomplished by aligning levees or floodwalls to tie into natural ground without including floodplain areas for which protection is not economically justified or without blocking all natural flood paths.

(3) The line-of-protection should be aligned to minimize requirements for pressure conduits to convey water from the interior area into the exterior. As noted below, use of such conduits is expensive and leads to less reliable, less robust facilities in most cases.

(4) The line-of-protection should be planned to avoid or minimize need for diversion of flood water out of the interior area. Again, this is an expensive solution to flooding problems, and often leads to a transfer of risk to other property.

(5) The line-of-protection should be aligned to minimize requirements for right-of-way (i.e., easements) that must be acquired as levees or floodwalls are constructed to protect the interior area from exterior flooding.

(6) The line-of-protection should be aligned to ensure that when the design capacity of the levee or floodwall is exceeded, overtopping into the interior area occurs in a planned manner, minimizing damage and permitting safe evacuation with minimum life safety risk.

c. Gravity Outlets. Gravity outlets are culverts, conduits, and other openings that permit gravity discharge of interior waters through the line-of-protection. No pumping is required.

(1) Outlet dimensions, locations, invert elevations, inlet and outlet configurations, gate configurations, and operating policy are determined initially through hydrologic and hydraulic engineering analysis, and adopted as project components for use during the planning process.

These technical components are refined though hydraulic design studies as the project moves toward final design configuration.

(2) Gravity outlets may be simply an extension of the natural drainage path or engineered interior storm water drainage system. In that case, gravity outlets should be located at or near the point at which the line-of-protection intersects the natural or existing conveyance system. Otherwise, an interceptor system with open channels or other storm drainage facilities will be required along the line-of-protection to capture and convey runoff to the outlet.

(3) Analysis and design considerations for gravity outlets are consistent with those of roadway culverts and similar facilities, as described in EM 1110-2-2902. The hydraulic analysis required may be completed by developing gravity outlet rating functions, with upstream conditions determined by the interior area drainage and downstream conditions determined by water levels in the exterior river, lake, or sea. For more detailed analysis or for complex configurations, computer programs such as HEC-RAS or HEC-HMS can be used (Table 4-3).

(4) A detention basin in the interior area may economically collect, store, and gradually release interior flood waters. If the detention basin outlet is the gravity outlet, the amount of risk reduction possible is a function of both basin and gravity outlet configuration. Analysis requires simulation and economic, environmental, and social impact evaluation of the integrated performance of the basin and the outlet throughout the range of expected interior and exterior events. (Note: Procedures for this are described elsewhere in this manual.)

(5) Gravity outlets are gated to prevent flow of water from the exterior stream into the interior area as the exterior stage rises above the interior elevation. Gates may be simple self-closing flap gates, or they may be more complex closure structures that require action to close. Normal operation policy will be to operate to release water through the outlet, following the reduction of exterior stage, maintaining a small positive head. The level of specificity required of an operation policy is dictated by the level of study. Planning studies require fewer details, while analyses to support final design will require details of interior and exterior elevations at which gates should be closed, when they should be reopened, and so on.

(6) If the existing condition for an interior area includes existing gravity outlet(s), operation policy should be obtained from the agency responsible for operating the interior drainage system. Modified operation criteria should be considered as flood risk reduction alternatives are formulated. Delaying or accelerating releases may further reduce risk, so this should be considered.

(7) Flood warning systems, flood monitoring and flow forecasting may improve gravity outlet operation. For example, if lag time between interior and exterior peak stages is a critical factor in the operation, the capability to anticipate future exterior or interior stage and operate proactively may reduce risk.

d. Detention Basins. As noted above, a detention basin (also called detention pond or detention area) is a facility that stores interior area runoff, releasing that to facilities that convey the water over or through the line-of-protection into the exterior water body.

(1) Detention areas may be natural or excavated sumps, vacant lots or areas, streets, and parks. Detention areas are commonly located adjacent to a gravity outlet or pumping station, but may also be remote from these facilities, connected by appropriately sized channels. Topography, geology, existing conveyance patterns, and land use govern choice of location. Detention basins may be dry, storing water only during floods, or wet, with a permanent pool.

(2) By design, the rate of release from a detention pond is less than the unregulated or natural rate of interior runoff. Thus, the gravity outlets or pumps required to move water away from the interior area can be smaller and less expensive as some portion of the water will be stored temporarily.

(3) A detention basin also will permit storage of water until exterior stages fall so that conveyance from the interior area is more efficient. For example, if interior runoff is stored until after the peak exterior stage, gravity outlets alone may be sufficient to convey large infrequent events. This will reduce interior risk by decreasing stage associated with rare events.

(4) A detention basin may also increase the reliability of the interior drainage system by providing additional time for appropriate operation before damaging water levels occur.

(5) Detention basins can be designed to be environmentally attractive and contribute to community social goals in urban areas when used as parks and open spaces during periods when not needed for runoff storage; often called multi-use detention basins. Care must be taken if dry basins are used for recreational or other activities to provide adequate warning for evacuation of the basin prior to storing water there.

(6) Management of the functional integrity of the detention basin by preventing development encroachment and subsequent loss of storage capacity is critically important. Local agency agreements should specify requirements for maintenance of detention basin functional integrity throughout the project life.

(7) Hydrologic and hydraulic analyses should assess flooding hazard in the interior area, consistent with methods identified in this manual. For that analysis, the impact of future development in the protected floodplain should be assessed, in particular in terms of additional storage requirements of the detention basin.

e. Pump Stations. Pumps are designed to lift storm water and other interior flows over or through the line-of-protection to the exterior river, lake, or sea if gravity flow is not feasible. Guidance on design and construction of pumping stations is provided in EM 1110-2-3102. Specific considerations for interior drainage are listed here.

(1) Pumps may be used for storm runoff, groundwater and seepage, water accumulated from overtopping of the line-of-protection, and combined storm water and sanitary sewage flows.

(2) The initial construction plus long-term operation and maintenance costs of the pumps and pump stations are significantly greater than costs associated with other measures identified herein. Therefore, pumps should be used for interior area drainage only if analysis of gravity outlets and detention storage demonstrates that the incremental cost of pumping is justified. For areas where the interior and exterior flooding is highly correlated, with high likelihood of blocked gravity outlets coincident with interior flooding, pumping may be the only means to significantly reduce interior flood risk. For areas where coincident high exterior stage and interior flooding are not likely, pumping facilities may not be required.

(3) As part of planning and subsequent design studies, the hydraulic engineer must determine and refine the location and alignment of pumps, their inlet structures, integrated ponds, and gravity outlets. Pumping stations commonly are located adjacent to the line-of-protection. The station should be aligned in a manner that enables direct flow patterns into the pump intake forebay from the conveyance channel or detention areas; thus, minimizing energy loss and reducing maintenance difficulties. The operational performance of the pump station must be evaluated for a range of flood events to ensure that sufficient water can enter the forebay without significant surges or frequent stop-starts of the station pumps.

(4) Studies must also determine the number, capacity, and types of pumps, their design efficiency, and their first or operation floor elevations. Pump capacity in urban areas is generally determined by the physical performance of the facility and its effect on flood risk management benefits, costs, and environmental and social factors. Station capacities in rural (agricultural-type damage) areas are more commonly based on economic optimization. First or operation floor elevations of pumping stations should be at least or above ground level to provide convenient access to equipment, to eliminate need for protection against groundwater, and to simplify the ventilation of the operation areas.

(5) Planning and design studies must identify elevations at which pumps will be turned on and turned off. Elevations at which pumps will be turned on and off should be set, if feasible, so that pumps may be operated once or twice annually for maintenance and testing.

(6) Gravity outlets commonly are included with a pump station; permitting gravity flow when exterior stages are lower than interior. The gravity outlets may be offset from pump intakes if direct flow access to both the pump and gravity outlets is insufficient. Planning and design studies must determine these locations and properties of the facilities.

(7) The consequence of exceeding pump design stage must be evaluated to assess the overall risk reduction and residual risk associated with pumping and with the entire interior drainage facility. In addition, off-site impacts must be assessed to ensure that risk is not transferred by raising exterior stages or otherwise adversely affecting operation of downstream gravity outlets or other facilities.

f. Intercepting Sewers or Channels. Intercepting pipes (sewers) or channels that connect two or more existing pipes or channels or overland flow areas in the interior floodplain convey flows behind the line-of-protection to gravity outlets, pumping stations, or pressure conduits for discharge through the line-of-protection.

(1) Interceptor systems are intended and must be planned and designed to reduce the cost of gravity outlets, pumping stations, and pressure conduits without adversely affecting the risk reduction provided by the interior drainage system.

(2) Sizes, locations, alignments, and other properties of these pipes or channels must be determined in the planning stage and refined further for design. Analysis for this must consider the operational performance of the entire system under the expected range of interior and exterior conditions.

g. Pressure Conduits. Pressure conduits include pipes and closed conduits that convey interior flood waters through the line-of-protection with internal pressure.

(1) Pressure conduits commonly serve as discharge lines for pumping facilities; however, permitting discharge from the interior area even when the exterior stage exceeds that in the interior.

(2) Sizes, locations, alignments, and other properties of these conduits must be determined in the planning stage and refined further for design. As with other features of the interior drainage system, analysis for this planning and design must consider the operational performance of the entire system under the expected range of interior and exterior conditions.

h. Seepage and Relief Wells. Levees and floodwalls are subject to seepage through their foundations and abutments, and that seepage may result in excess hydrostatic pressure or uplift pressures beneath the structure. Relief wells may be required as a component of the line-of-protection and interior drainage system.

(1) Relief wells are described by EM 1110-2-1914 as "controlled artificial springs that reduce pressures to safe values and prevent the removal of soil via piping or internal erosion may be required to relieve this pressure". The proper design, installation, and maintenance of relief wells are essential elements in assuring their effectiveness and the integrity of the protected interior area.

(2) Seepage water collected with relief wells must be managed to avoid flood damage. Drainage paths (natural flow paths, channels, or conduits) between relief wells or seepage collection channels and the interior drainage facility may be required to reduce ponding due to seepage. Those paths should be included as a component of the interior drainage system.

6-3. Structural Measures Remote from Line-of-Protection.

a. Types of measures. Structural measures remote from the line-of-protection include:

(1) Conveyance channels

(2) Diversions

(3) Detention basins

(4) Interior levees and walls

b. Conveyance Channels. Conveyance channels reduce flood risk at locations remote from the line-of-protection by collecting and efficiently conveying runoff and other interior waters to gravity outlets, pumping stations, and pressure conduits.

(1) If possible, channels should be planned, designed, and constructed to follow natural drainage and conveyance routes. When this is not possible, channels should be located near and parallel to the line-of-protection.

(2) Interior area channels may be required with detention basins to connect with gravity outlets or pumping stations. Guidance for analysis of channel hydraulics (EM 1110-2-1416) and for design of channels (EM 1110-2-1601) should be followed.

(3) Channels on the exterior side of the line-of-protection may be required to connect outlet works of gravity or pressure conduits or pumping stations to the river, lake, or sea.

(4) The planning task, and subsequently the design task, is to determine appropriate size, location, and properties of the channel system.

c. Diversions. Diversions within the interior area may be included to transfer all or portions of the runoff from one location to another. For example, runoff may be split so that some portion is stored in a detention basin, while the remainder is discharged through a gravity outlet or pumped.

(1) Diversions may be planned, designed, and constructed to collect flow for pressure conduits, to transfer flow out of the basin (reduce the contributing area), and to collect flow from areas to gravity outlets and pumping stations, thereby enabling fewer facilities.

(2) Diversions may be designed to alter permanently the interior floodplain conveyance system for the entire range of flows or to operate only for discharges above (and below) certain values. Diversions may be designed to bypass certain damageable property, conveying flow around damage centers.

(3) Diversions may be uncontrolled or operated as part of a coordinated system. If a diversion is to be operated with gates or other control structures, design of those features is subject to usual criteria for flood management structures, including, for example, the need for operation and maintenance manuals, agreements with the local maintaining and operating agency for inspection, and so on.

d. Detention Basins. Detention or storage basins remote from the line-of-protection may be included to manage runoff behind the line-of-protection.

(1) Interior ponds have characteristics similar to those of detention basins adjacent to the line-of-protection. They capture and store runoff from the interior floodplain, then release that water so that regulated peaks are less than those without regulation. This reduces flood risk by reducing hazard.

(2) Interior area detention basins may be natural sinks or oxbow lakes in bottomlands or excavated sumps, or they may be formed by levees. Hillside or bluff basins commonly are conventional reservoirs. The detention basins may be either dry or wet, they range from smaller detention ponds to hillside reservoirs, and they may be distributed throughout the interior area or located at a single site to provide regional detention before floodwaters reach facilities at the line-of-protection.

(3) Interior area basins located away from the line-of-protection will regulate flow to reduce flooding in the interior floodplain. A pond or basin also reduces flows into the gravity outlets, sumps, pumps, and pressure conduits at the line-of-protection. Thus an interior pond properly sized and located may reduce the size and cost of facilities at the line-of-protection.

(4) Interior detention basins also may retain sediment that washes off of hillside or bluff areas. Thus the basin may reduce deposition elsewhere in the interior floodplain or in the sump, outlets, or pump at the line-of-protection. Removal or maintenance of sediment deposition should be considered in the lifecycle cost of this measure.

e. Interior Levees and Walls. Interior levees and walls along interior floodplain conveyance channels may be implemented as local interior protection features

(1) These barriers to channel overflow commonly are lower in height than levees that provide the line-of-protection by separating the interior floodplain from the exterior channel. Consequently, capacity exceedance or failure is less likely to cause catastrophic loss of life or damage to property. Nevertheless, the risk reduction, including analysis of residual risk associated with the levees or walls must be completed as a component of planning and design studies.

(2) If the interior levees or walls are of sufficient height, and damage potential from failure is great, they are considered the same as the main line levees or walls.

(3) The interior levees may create secondary interior flooding problems that must be considered.

(4) Flood forecasting and emergency preparedness plans should be an integral part of implementation of interior levees and walls; thus, managing the residual risk associated with the measures.

6-4. Nonstructural Measures.

a. General. ER 1105-2-100 requires consideration of nonstructural measures. These are measures that reduce flood risk by limiting the exposure and consequence of flooding.

(1) Nonstructural measures include temporary or permanent flood proofing, relocation of structures, flood warning systems (with monitoring, predicting, and response), and regulation of floodplain uses.

(2) The measures may be implemented independently or in combination with structural measures.

(3) Nonstructural measures may reduce risk for both existing and future property within the floodplain.

b. Nonstructural measures that modify damage potential of existing floodplain property include:

(1) Temporarily or permanently flood proofing with closures, seals, and/or backflow prevention devices.

(2) Raising (lifting and repositioning) existing structures in place.

(3) Constructing small earthen dikes or walls around individual structures or small groups of structures.

(4) Temporarily relocating occupants and property away from at-risk locations.

(5) Permanently relocating occupants and property away from at-risk locations, with or without demolition of the property, if other appropriate uses are identified.

(a) These measures commonly are implemented on a localized scale (such as a neighborhood).

(b) These measures eliminate damage until design limits are exceeded. For example, if a structure is raised so that its first floor elevation is above the water surface elevation associated with a specified exceedance probability, damage will still be incurred if a larger event occurs. Thus, residual risk must be evaluated and considered.

(c) These measures, applied to individual structures or small groups of structures, commonly are less disruptive environmentally than structural alternatives.

(d) The measures do not reduce damage to vital services (such as water, gas, and power) or to streets, bridges, and landscaping. In most cases, the measures reduce only slightly the social disruption caused by flooding.

(e) Floodproofing, raising, and relocating often are less costly than structural measures when only a few structures are involved.

(f) A trade-off exists as nonstructural measures are implemented, and this must be assessed during planning. For example, if a small group of low-lying structures near the line-of-protection are protected with small walls, a larger sump at the line-of-protection may be feasible. That, in turn, may permit use of smaller gravity outlets or elimination of pumps. Economic, environmental, and other social impacts of these decisions must be assessed as the interior system is planned along with the line-of-protection.

c. Nonstructural measures that manage future development can reduce flood risk by ensuring appropriate use of floodplain lands.

(1) Management of future development reduces losses by requiring that floodplain development and activities be appropriate, given the flood hazard. For example, development in frequently inundated areas can be restricted to require construction of new structures above a certain level or construction with certain waterproof materials.

(2) Future development, and hence future flood risk, can be controlled with zoning ordinances, building codes and restrictions, taxation, purchase of land in fee, or purchase of a flood easement. Structures not precluded from floodplain locations by these measures may locate on the floodplain if constructed and maintained to be compatible with the recognized flood hazard. (NOTE: That it is not the responsibility of the federal government to impose these ordinances, codes, etc. However, planning activities can better inform local agencies of the benefits of such actions.)

(3) Regulatory actions and land acquisition can also bring about new use of the floodplain. The measures are attractive from the perspective of managing development to reduce the future damage potential of the area and use of the floodplain for compatible purposes.

(4) Measures that manage future development are generally compatible with implementation of other structural and nonstructural measures. For example, regulatory actions may be incorporated as part of the agreements with local agencies or the local sponsor; local

policies to preserve the storage and functional integrity of detention basins over the life of the project may be required of local cost-sharing partners as part of the cost-sharing agreement.

d. A complete flood warning system may reduce flood risk by providing notification in advance of flooding, providing time for actions that save lives and protect property.

(1) A complete flood warning system includes remote monitoring instruments, data transmitting and receiving devices, evaluation and threat detection resources, notification procedures, emergency response plans, and recovery procedures and plans. Together, these can reduce flood risk minimize social disruption, and guide recovery and reoccupation of flooded areas.

(2) The risk reduction attributable to a flood warning system is a result of the advance notice that permits evacuation of the floodplain, temporary protection of property within the floodplain, closure of openings in the line-of-protection, and so on.

(3) Preparedness plans should define action thresholds for closure of line-of-protection openings, including gravity outlet gates and roadway or railway openings. Preparedness plans should also define thresholds and procedures for initiating and conducting flood fighting and evacuation.

(4) In general, a flood warning system alone is not a complete alternative to other structural or nonstructural alternatives. However, it is an effective approach to managing residual risk with certain measures (such as levees), and it provides information that is required for and an integral part of other measures (such as pumps that must be operated considering future inflows).

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CHAPTER 7

Analysis of Residual Risk and Resiliency

7-1. Overview. This chapter provides guidance on the analysis of residual risk and resiliency, consistent with procedures described in EM 1110-2-1619.

7-2. Analysis Purpose. Risk analysis data generated to analyze an interior drainage project can be used to determine the best combination of interior drainage facilities. As described in Section 2-2, the interior facilities are designed to alleviate flooding which results from the addition of line-of-protection features. The interior area can be divided into an impacted and a non-impacted area. The impacted area is the area where flooding is created by the pond resulting from the addition of the risk reduction measures and the reach of stream or creek above the pond that is now impacted by backwater created by the pond. The non-impacted area is the area upstream of the interior pond (Figures 7-1 and 7-2) and is not impacted by the interior pond or backwater generated from the interior pond. In some cases, both areas need to be evaluated. An

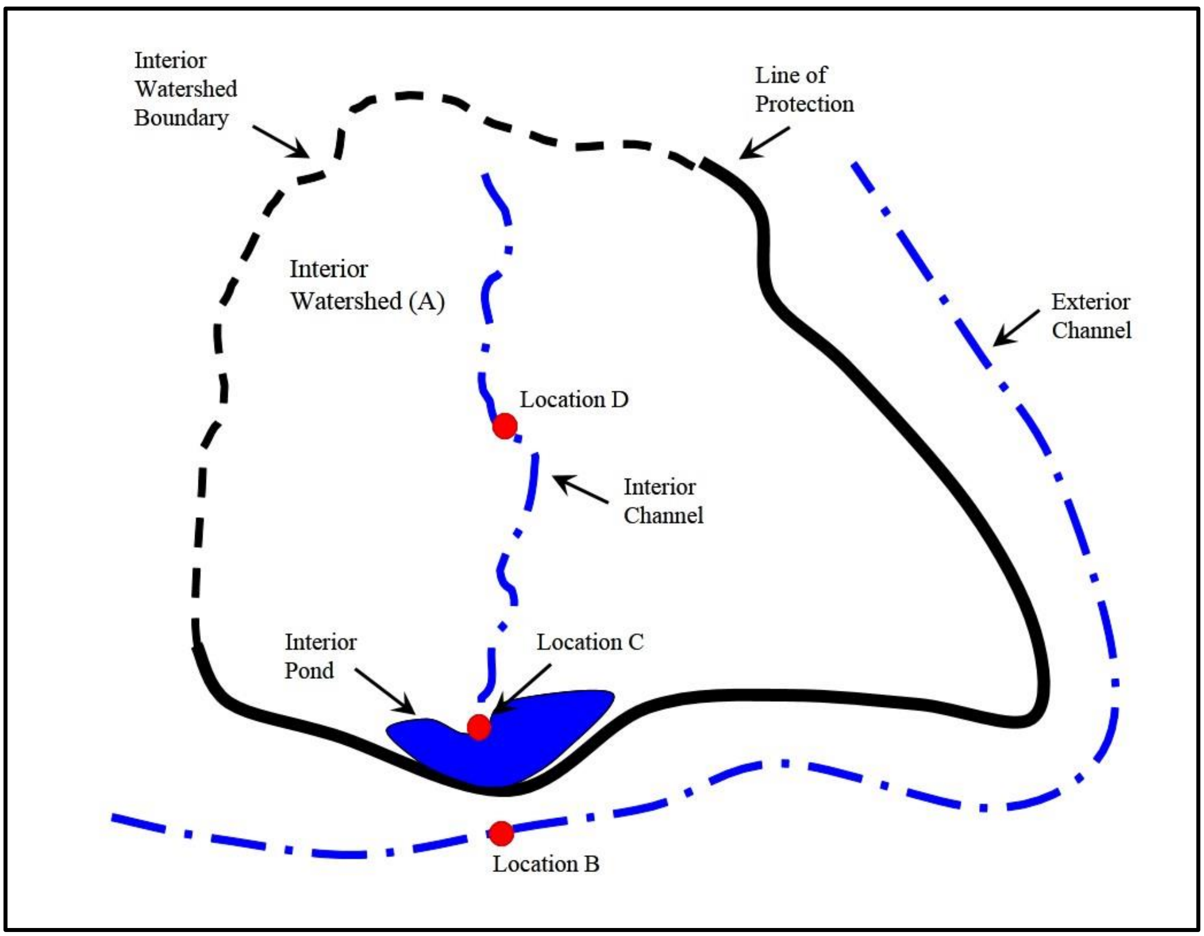

Figure 7-1. Plan view of interior system: line-of-protection prevents flooding from exterior channel but also obstructs natural drainage of interior watershed.

example would be if a project is proposing an upstream diversion or storage to help alleviate downstream interior flooding. Plan formulation for interior projects will follow the general plan formulation discussion in alignment with ER 1105-2-100 and ER 1105-2-101.

7-3. Data Requirements.

a. Stage-Damage Function with Uncertainty. The development of the stage-damage function is described in EM 1110-2-1619. A stage-damage function models economic vulnerability of flooding as a function of flood stage for each element of inventory in the floodplain. Thus, use of a stage-damage function presumes that the critical variable in predicting damage is the floodwater surface elevation at the property.

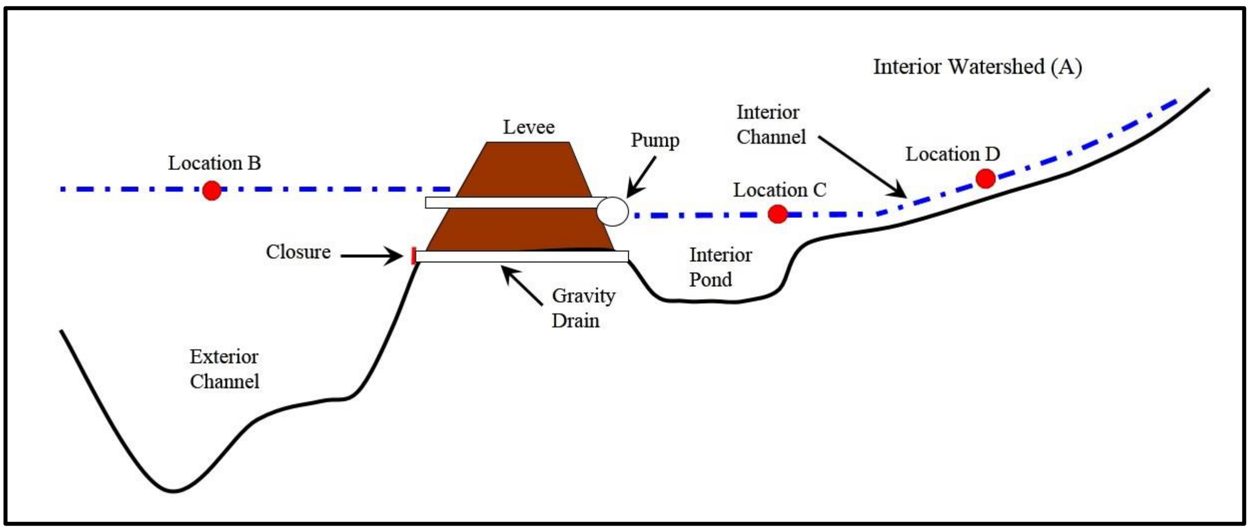

Figure 7-2. Profile view of simple interior system: state of system at Location C is influenced by interior runoff (Watershed A) and exterior stage at Location B.

(1) A stage-damage function characterizes the vulnerability of flooding of the inventory in a floodplain. The function may represent loss for a single element of the inventory, such as a single residence and its content, or the function may represent the aggregated loss for groups of items. The function may represent tangible (direct and/or indirect) loss and/or intangible loss.

(2) The total uncertainty about the stage-damage function is a combination of the errors, inaccuracies, or lack of knowledge regarding items listed in paragraph 6-3 of EM 1110-2-1619. (For risk analysis accounting for uncertainty, a model of the uncertainty about the best estimate of the stage-damage transform must be selected.) Historically, a wide range of distributions have been used, including uniform, triangular, normal, log-normal, and asymmetrical distributions. Robust documentation is absent that would justify the use of either very simple distributions or the use of more complex distributions. The uncertainty in most parameters is probably most often best described by normal or log-normal distributions.

b. Discharge-Probability Function with Uncertainty. The development of the discharge-probability function is described in EM 1110-2-1415, EM 1110-2-1417 and EM 1110-2-1419. Guidance on means for determining the equivalent years of record is described in EM 1110-2-1619.

(1) The without-project, discharge-probability function would describe the flow in the main river at an index point, such as Location B in Figures 7-1 and 7-2. Depending on the possible risk reduction measures that are envisioned it may also be necessary to generate discharge-probability functions in the upstream area of the Interior Watershed away from the influence of flow in the exterior channel, such as at Location D.

(2) The with-project conditions discharge frequency function would reflect changes in the function related to possible risk reduction projects upstream of any index point. Should there be no risk reduction measures, such as detention or diversion, that would impact the function then use of the without-project, discharge-probability function could be warranted to represent with-project conditions.

c. Stage-Discharge Function with Uncertainty. The development of the stage-discharge function is described in EM 1110-2-1619.

(1) The stage-discharge function is used to transform the flow selected from the flow-probability function into a stage value to compare to a specific elevation, such as top of levee. The stage-discharge function and its Uncertainty for index locations in the non-impacted interior area and in the main river, Location B in Figures 7-1 and 7-2, should be developed using the guidance in EM 1110-2-1619. The non-impacted area is described as the area upstream of any influence of the Interior Pond such as Location C in Figures 7-1 and 7-2. The development of the uncertainty about the stage-discharge function is described in EM 1110-2-1619.

d. Stage-Probability Function with Uncertainty. The development of the stage-probability function is described in EM 1110-2-1619.

(1) The general data required for risk analysis is a peak flow-probability function with uncertainty; stage-flow function with uncertainty; stage-damage function, with uncertainty and an elevation of concern. However, there can be times when risk analysis results are needed in a location that is influenced by backwater. In this case, the stage-flow function and peak flow-probability function would not be representative of the location since the backwater influences can result in a stage that could represent multiple flow values, depending on backwater conditions. In this situation, the use of a peak stage-probability function to replace the stage-flow and peak flow-probability functions would be appropriate. However, care must be taken when making this decision. The uncertainty about the peak stage-probability function only represents the uncertainty in stage. The flow uncertainty is not considered. Both stage uncertainty and flow uncertainty are considered when the stage-flow and peak flow-probability functions are used.

7-4. Procedure Overview.

a. General. This section will provide general discussion and guidelines on performing analyses for various types of conditions. It will not discuss procedures in detail but will provide a roadmap on what is needed to perform risk analysis for an interior area. This section is divided into two parts. The first is how to analyze the impacted area and the second is how to analyze the non-impacted area. It may not always be necessary to analyze the non-impacted area. The basic description on performing risk analysis is in EM 1110-2-1619.

b. Existing Without-Project Conditions. The first step is to analyze the existing without-project conditions. This analysis provides the baseline.

(1) Maximum Ponding Inundation Boundary. As described earlier in Chapter 3 and depicted in Figure 3-2, the damages resulting from the main stem river and interior area for without project existing conditions must be computed. This can be accomplished by developing a hydraulic model of the main stem river and tributary area. It will be necessary to independently develop the flow-probability curves and corresponding hydrographs (if performing unsteady analysis) for the mainstem river flow boundary conditions. In conjunction with the mainstem river flows, concurrent flows will be used in the interior steam. The inundation boundary should be developed for the interior area for each frequency, which causes damage, and used to develop the stage-damage curve for damage from the main stem river.

(a) Similarly, the hydraulic model can be used to determine damages resulting from high flows on the interior stream. It will be necessary to independently develop the flow-probability curves and corresponding hydrographs (if performing unsteady analysis) for the interior stream flow boundary conditions. In conjunction with the interior stream flows, concurrent flows will be used in the main stem river. The inundation boundary should be developed for the interior area for each frequency, which causes damage, and used to develop the stage-damage curve for damage from the interior stream.

(b) When aggregating the damages, it is important to follow the flood damage evaluation concepts described in Section 3-2. All damages will be aggregated to the index point(s) chosen to represent the project area.

(2) Flow-Probability Function with Uncertainty. The flow-probability function can be developed from multiple sources. EM 1110-2-1619 discusses the development of the flow-probability function and its uncertainty.

(3) Stage-Damage Function with Uncertainty. Stage-damage functions required for risk analysis of an impact area may be developed by taking the following steps:

(a) Define the spatial extent of the impact area,

(b) Identify structures, contents, infrastructure, agricultural crops, and any other at-risk elements to be included in the risk analysis,

(c) For each element of exposed inventory, establish the reference elevation,

(d) For each element, estimate the economic value,

(e) For structures, this value is the depreciated replacement cost minus land value,

(f) For each element, adopt or develop a depth-damage model,

(g) Transform the depth-damage model of each element to a stage-damage function, using the reference elevation from the third step and the economic value from the fourth step,

(h) If appropriate, aggregate the stage-damage functions.

(i) The common approach to developing a stage-damage function for a single element in an impact area is to select, calibrate, and use a standardized depth-percent-damage function. These functions, one of which is shown in Figure 6-2 of EM 1110-2-1619, commonly shows the economic value of damage, expressed as a fraction or percentage of the total value of a property, as a function of the water depth at the structure.

(4) Stage-Flow Curve with Uncertainty. The hydraulic model used to develop the inundation boundary would be used to develop the stage-flow curve for the index point(s) chosen to represent the study area.

(5) Stage-Probability Function with Uncertainty. There are times when a stage-probability curve, with uncertainty needs to replace the flow-probability and stage-flow curve in an analysis. This would be in conditions where the same flow could result in different stages at the index location. This would be mainly due to downstream conditions which generate backwater. The analyst must decide if the stage-probability curve is appropriate. Refer to EM 1110-2-1619 for a discussion on uncertainty for the stage-probability curve.

(6) Compute Existing Without-Project EAD and Performance Indices. Apply the HEC-FDA software with appropriate inputs developed above to determine the EAD and the performance indices computed by HEC-FDA. HEC-FDA provides the capability to perform an integrated hydrologic engineering and economic analysis during the formulation and evaluation of flood risk management plans. HEC-FDA is designed to assist study members in using risk analysis procedures for formulating and evaluating flood risk management measures (EM 1110-2-1619; ER 1105-2-101; HEC, 2015b).

c. With-Project Conditions. The analysis of with-project conditions provides the basis for comparing project alternative benefits to without-project conditions and to each of the project conditions. This provides input for the determination of the NED project alternative. These analysis steps would be performed for each project alternative. Depending on the alternative it may be possible to use some, or all, of the results of the analysis of a previous alternative.

(1) Maximum Ponding Inundation Boundary. As described earlier in Chapter 3 and depicted in Figure 3-2, the damages resulting from the main stem river and interior area for with project existing conditions must be computed. This can be accomplished by developing a hydraulic model of the main stem river and tributary area which represents the project alternative including any interior drainage facilities. It may be necessary to develop the flow-probability curves and corresponding hydrographs (if performing unsteady analysis) for the main river flow boundary conditions. However, if flows for project conditions are the same as for without-project conditions then it would not be necessary to develop new flow data. Therefore, if with-project conditions include measures that alter flows from the without-project conditions then these new flows must be used. In conjunction with the main river flows, concurrent flows will be used in the interior stream. The interior stream flows are subject to the same-project/with-project comparison discussed for the main river. The inundation boundary should be developed for the interior area for each frequency, which causes damage, and used to develop the stage-damage curve for damage from the main river.

(a) Similarly, the hydraulic model can be used to determine damages resulting from high flows on the interior stream. Using the logic described in the previous paragraph for the main river, it may be necessary to independently develop the flow-probability curves and corresponding hydrographs (if performing unsteady analysis) for the interior stream flow boundary conditions. In conjunction with the interior stream flows, concurrent flows will be used in the main river. The inundation boundary should be developed for the interior area for each frequency, which causes damage, and used to develop the stage-damage curve for damage from the interior stream.

(b) When aggregating the damages it is important to follow the flood damage evaluation concepts described in Section 3-2. All damages will be aggregated to the index point(s) chosen to represent the project area.

(c) This analysis should include events that exceed the capacity of the line-of-protection.

(2) Flow-Probability Function with Uncertainty. The flow-probability function developed for with-project conditions should represent any changes to flow values that would be caused by any risk reduction measures planned as part of the alternative. For example, if flows are diverted away from the project area then the flow-probability function should represent this change. EM 1110-2-1619 discusses the development of the flow-probability function and its uncertainty.

(3) Stage-Damage Function with Uncertainty. The stage-damage function used to evaluate with-project conditions is typically the same function as the without-project condition as described in paragraph 7-4,b(3). This basic assumption then maps the changes to with-project condition in flow-probability, stage-flow, and/or system stability into a with-project damage probability function from which a with-project EAD is computed.

(4) Stage-Flow Curve with Uncertainty. The hydraulic model used to develop the inundation boundary would be used to develop the stage-flow curve for the index point(s) chosen to represent the study area.

(5) Stage-Probability Function with Uncertainty. There are times when a stage-probability curve, with uncertainty needs to replace the flow-probability and stage-flow curve in an analysis. This would be in conditions where the same flow could result in different stages at the index location. This would be mainly due to downstream conditions which generate backwater. In the case of the project analysis, the analyst must decide if the stage-probability curve is appropriate to represent conditions resulting from the project. Refer to EM 1110-2-1619 for a discussion on uncertainty for the stage-probability curve.

(6) Compute Existing Without-Project EAD and Performance Indices. Apply HEC-FDA with appropriate inputs developed above to determine the EAD and the performance indices computed by HEC-FDA.

(7) Repeat steps 1 through 5 for each project alternative.

d. Non-Impacted Areas. Analysis of the non-impacted area may be necessary if a project measure (diversion, storage, etc.) necessary to mitigate interior ponding would also reduce flooding in this area. This would be flooding caused from flows in the interior stream as opposed to flooding caused by the ponding resulting from the line-of-protection construction. If this is the case then the non-impacted area should be analyzed for both without-project and project conditions.

e. Identify NED alternative. Description for identification of the NED alternative can be found in ER 1105-2-101, and ER 1105-2-100.

7-5. Reporting Requirements.

a. General. Refer to ER 11105-2-101 and EM 1110-2-1619 for guidance on displaying and reporting risk and uncertainty results.

b. Planning Study Requirements. The goal in presenting the results of risk and uncertainty analyses in USACE studies is to facilitate comparison of the effectiveness of alternative plans in terms of providing economical, safe, and predictable protection; solving the problem; and taking advantage of the opportunities identified. The report documenting a USACE planning study must identify the monetary and non-monetary benefits and costs of the alternatives; identify differences among the alternatives; and describe fairly the uncertainty about any benefits, costs, and risk indices. ER 1105-2-101 requires that the estimate of net benefits and benefit/cost ratio be reported both as a single expected value and on a probabilistic basis for each planning alternative. The probability that net benefits are positive and that the benefit/cost ratio is at or above 1.0 will be presented for each planning alternative. ER 1105-2-101 provides several examples of how this information can be reported.

c. Design Documentation Requirements. The Design Documentation Report (DDR) is a record of final design efforts after the feasibility phase. A DDR is required for all engineering design products. The DDR provides the technical basis for the plans and specifications and serves as a summary of the final design. The DDR covers the preconstruction engineering and design phase and the construction phase of the project. Engineering performance (AEP, CNP, etc.) of the project reported in the planning phase should be confirmed/updated in the design phase and reported per ER 1105-2-101 and EM 1110-2-1619.

CHAPTER 8

Period-of-Record Example

8-1. Purpose. This example describes, with a case example, the period-of-record analysis procedure for performing hydrologic studies of a leveed interior area. The example emphasizes concepts in a feasibility study setting. The reader should be familiar with the material in Section 4-5 prior to studying this example.

8-2. General Study Background. USACE is performing a feasibility study of remedies for interior flooding of the Jones Drainage and Levee District, an agricultural area in the Smith River Valley. The area is protected from direct river flooding to a two-percent chance exceedance frequency event by a main levee and a tie back levee (Figure 8-1). The interior area consists of 5,000 acres in the Smith River floodplain and receives runoff from a ten square mile watershed.

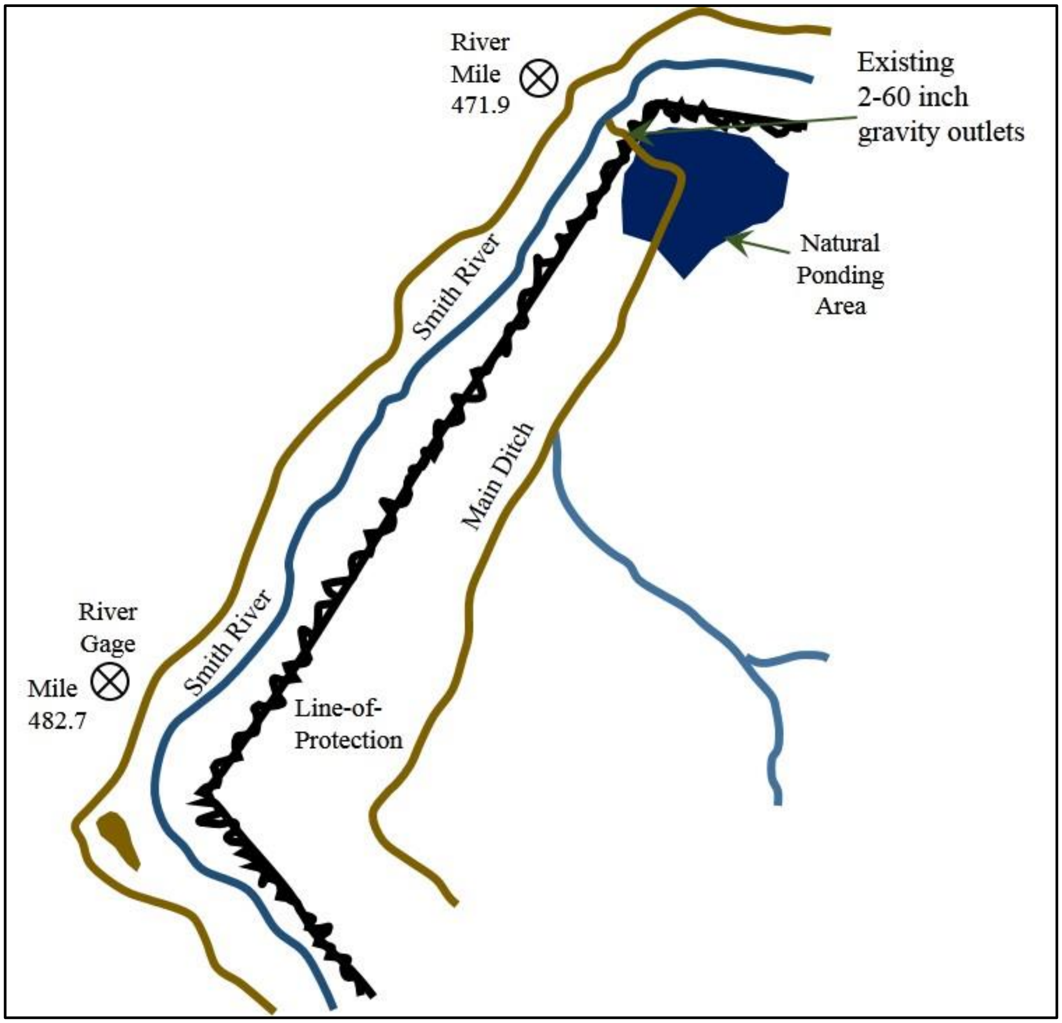

Figure 8-1. Study area map.

Runoff is conveyed through the interior area by a network of lateral ditches and main channels. The only outlet for interior runoff is an existing gravity outlet comprised of double sixty-inch diameter culverts through the line-of-protection.

a. During large events, runoff from the interior area ponds behind the levee. Agricultural crop damage has resulted from ponding of local runoff adjacent to the line-of-protection. Damage occurs during prolonged periods of blocked gravity outflow caused by high exterior river stages. Flooding commonly occurs in the spring months. Approximately one-half of the area has been inundated three times during the past ten years.

8-3. Study Strategy.

a. Reconnaissance level studies found that significant flood damage potential existed in the interior areas and that it is justified to study alternative flood loss reduction plans. These plans include combinations of modifications to ditches, channels, and gravity outlets, and the installation of pumping facilities. Period-of-record analysis procedures are used to develop hydrologic data for agricultural flood damage assessments, optimal sizing of additional gravity outlets and pumping facility capacities, and selection of pump operating criteria. Data requirements and hydrologic analysis procedures used in the plan formulation portion of the study are described in Section 4-5.

b. Period-of-record analysis procedures are applicable because of the availability of long-term precipitation and exterior stage data. Flood damage evaluations may be computed directly from each historic event by accounting for season, magnitude, and duration of the event. Annual pump operation times may also be directly calculated.

c. The period-of-record analysis is performed for with and without proposed improvement for existing and future conditions. The existing condition minimum facility is assumed as the gravity outlet is presently in place. The formulation strategy involves initial evaluations of additional gravity outlet capacity (ultimately found not feasible) and subsequent analysis of various pumping facility sizes. A period-of-record assessment is performed for the existing conditions without a proposed improvement project, and for each gravity outlet and pumping facility size. Since no change in the agricultural area is projected throughout the project life, future hydrologic conditions are the same as existing conditions.

8-4. Hydrologic Analysis Methods.

a. General. Analysis of the interior area is based on data requirements for period-of-record precipitation-runoff response parameters, ponding area geometry, seepage, gravity outlet and pumping capacities, and exterior stage conditions. A HEC-HMS model was used to simulate the precipitation-runoff response from the interior area as well as the drainage analysis, route runoff from the interior area through the gravity outlet and pump facilities. The hydrologic simulation was performed at a one-hour interval for the 50-year period-of-record selected for analysis. The resulting interior stage-hydrographs are used in damage calculations. The formulation strategy analyzed several sizes of gravity outlets and pumping station capacity.

b. Historic River Stage Data. Historic river stage data were required at the gravity outlet and proposed pumping facility location (River Mile 471.9) to perform the period-of-record coincident routings through the line-of-protection. The period-of-record stage data were developed from the historic streamflow record of the nearby stream gage (River Mile 482.7) using an elevation-discharge relationship (Table 8-1). The elevation-discharge relationship was developed using a hydraulics model of the Smith River. The distance between the stream gage and the gravity outlet / pumping facility location could be long enough to require additional analysis (the incremental drainage area would produce significant runoff). Runoff from the incremental drainage area could be added to the stream gage data by developing a precipitation-runoff model or by developing a relationship between drainage area and flow (incorporating other hydrologic variables as necessary).

Table 8-1. Exterior elevation-discharge relationship.

Exterior Stage at Gravity Outlet River Mile 471.9 (ft)	**Discharge (cfs)**
361.2	0
363.1	1,000
365.1	10,000
367.0	50,000
369.0	100,000
370.9	150,000
372.7	180,000
382.2	300,000
391.8	350,000

c. Precipitation Data. Due to the size of the interior area, an hourly time-interval was selected as appropriate for this period-of-record analysis. An hourly time-interval allows for adequate definition of the interior area hydrograph. Review of exterior streamflow and hourly precipitation data obtained from the U.S. Geological Service (USGS) and NOAA, respectively, indicate an analysis period of fifty-years. This period-of-record length is considered adequate for the study area. Hourly precipitation data were obtained from NOAA website, http://www.ncdc.noaa.gov/oa/ncdc.html, at three nearby rain gages and used to develop a precipitation distribution pattern for each subbasin in the HEC-HMS model. This is accomplished by weighting the respective contribution of each rain gage based on the distance of the gage from the centroid of each subbasin.

d. Precipitation-Runoff Analysis. A continuous precipitation-runoff model was developed for the interior area. The interior area was subdivided into three subbasins with relatively similar land use and soil characteristics. Hydrologic parameters, needed to model processes like infiltration and overland flow, were estimated using geographic datasets and regional regression equations. For example, regional regression equations were used to estimate the travel time and storage coefficient for the Clark unit hydrograph method using physical characteristics of the watershed. The Soil Survey Geographic (SSURGO) database for the study area was obtained from the Natural Resource Conservation Service (NRCS) website. This database contains both spatial and tabular soils data that was used to estimate subbasin average parameters. An important process for continuous simulation is the removal of water by vegetation from the soil. Pan evaporation data, available from the same NOAA website used to gather precipitation data, was used to model potential evapotranspiration in the model.

e. Seepage. A secondary inflow or outflow into or out of the ponding area is seepage, which occurs through or under the line-of-protection. A relationship of seepage rate versus the differential head between the interior pond and exterior river is estimated based on pumping tests of interior relief wells installed for levee stability and estimates by foundation engineers obtained from similar studies. Table 8-2 shows the seepage rate versus head relationship.

Table 8-2. Head versus seepage relationship.

Head (Difference in Exterior River Stage and Interior Pond Stage) (feet)	**Discharge (cubic feet per second)**
0.0	0.0
2.5	15.0
12.5	80.0
23.5	155.0
31.5	180.0

f. Interior System Characteristics. The physical characteristics of the interior system defined for the analysis are the elevation-storage relationship for the ponded area, gravity outlets, and pumping stations. Their locations are shown in Figure 8-1. The ponded area is defined by an elevation-storage relationship. The major damage to crops in the interior area occurs from ponding in this area. The double sixty-inch gravity outlet conveys water from the interior area through the line-of-protection. The outlets function only for a positive head condition (interior pond elevations are higher than the exterior river elevation). The hydrology model was used to compute flow through the gravity outlets. Alternative pumping facility capacities are analyzed as part of the feasibility study. The pump location is adjacent to the ponding area. The pump head-capacity relationship is based on information supplied by pump manufacturers (Table 8-3). Pump on and off elevations are based on the proposed plan of operation.

Table 8-3. Pump head-capacity relationship alternative.

Pump Head (Total Head, Including Equipment Loss) (feet)	**Discharge (cubic feet per second)**
0.0	300.00
15.000	285.00
18.000	277.50
20.000	262.50
23.000	225.00
25.000	180.00
28.000	75.000
30.000	0.0

g. Interior Ponding Routing. The hydrologic model was first used to compute the precipitation-runoff for the interior drainage basin. Then the model was used to route the computed hydrograph through the interior ponding area to the exterior river. As mentioned, the interior area was modeled using an elevation-storage relationship, gravity outlets, seepage, and a proposed pumping facility. Inflow into the interior pond includes runoff from the interior area and seepage. Outflow may result from gravity outlets, when the exterior river elevations are lower than the interior ponding stage, and from pumping. The volume of inflow that exceeds outflow is stored in the ponding area, thus, causing the stage in the interior pond to increase.

h. Calibration Procedure. The period-of-record hydrologic simulation model is calibrated to historic high water marks and the observed frequency of flooding at roads, bridges, structures, and landmarks located in the ponding area. Adjustments are made to model parameters that affect peak stages and runoff volume.

i. Results. Figure 8-2 shows annual maximum stage for the interior pond from the period-of-record analysis for both the minimum facility and minimum facility plus pump simulations. These results show how the pump alternative is able to reduce the peak stage in the interior pond. Notice how the minimum facility plus pump data diverge from the minimum facility only alternative at an interior stage of 382 feet. This occurs because the pump is configured to turn on when the water elevation in the interior pond reaches 382 feet.

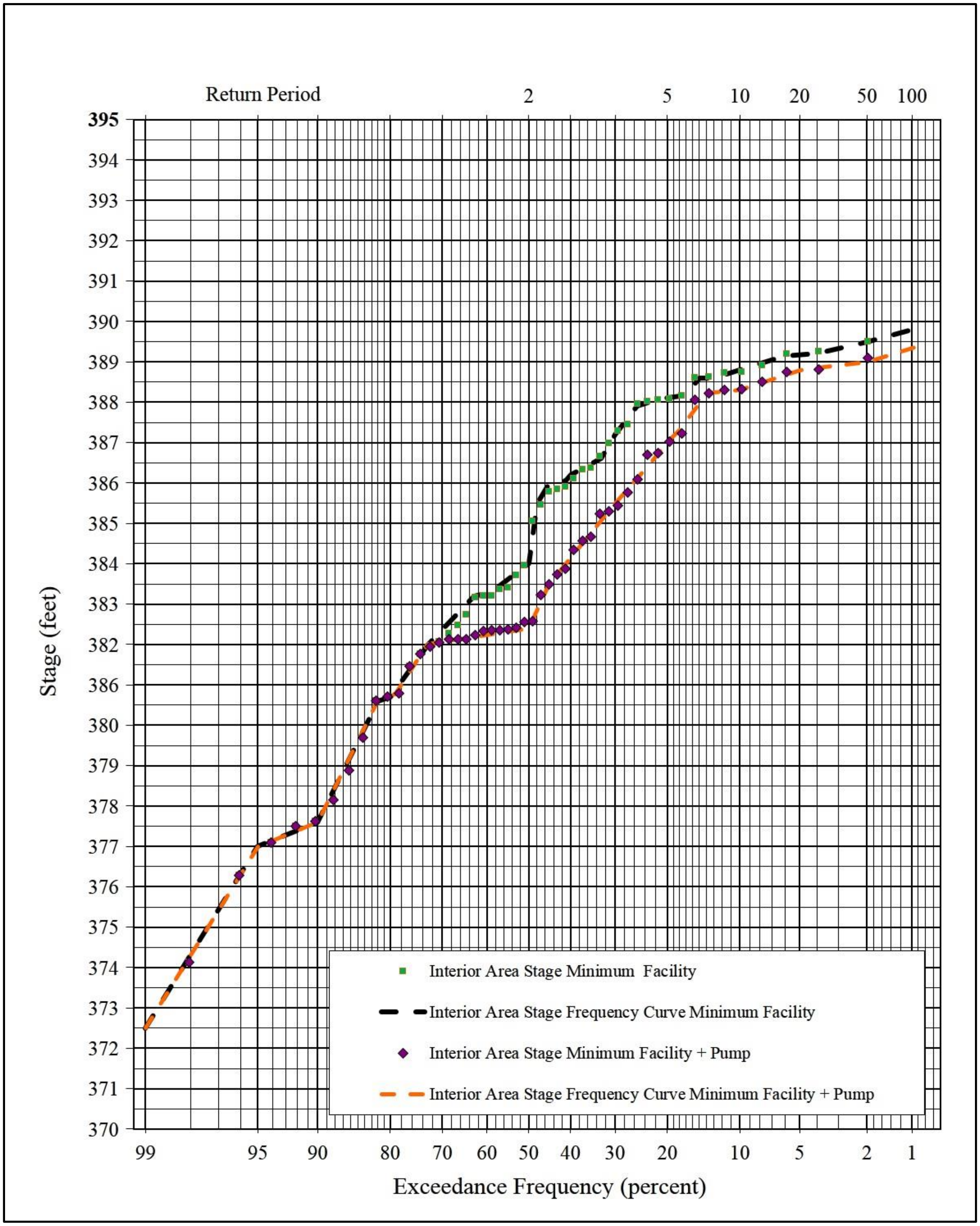

Figure 8-2. Comparison of minimum facility and minimum facility plus pump simulations.

8-5. <u>Summary</u>.

a. The period-of-record method of analyzing the coincident interior flooding of leveed or walled areas simulates the physical process of inflow, outflow, ponding area storage and outflow over time. The procedure is especially applicable to analysis of interior systems where the primary concern is at a ponding area adjacent to the line-of-protection where periods of high exterior stage blocks outflow from the interior area.

b. The example described here is typical of a single pond analysis for an area adjacent to the line-of-protection. Exterior stages are determined by transfer of a historic record from a nearby stream gage. A hydrologic model is used to simulate runoff from the interior watershed using available precipitation data. An additional inflow simulated by the hydrology model is seepage from the exterior river. Finally, the hydrology model is used to store water in the interior pond and route it to the exterior river using gravity outlets and pumps.

c. The flood damage reduction measure formulation process requires analysis of various sizes of gravity outlets and pumping facilities. Alternative gravity outlet invert elevations and pump on-off operation conditions are also evaluated. These assessments require additional analyses of the alternatives for the period-of-record.

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CHAPTER 9

Coincident Frequency Example

9-1. Purpose. This exhibit describes a case example of the coincident frequency method of performing hydrologic studies for a leveed interior area. The example emphasizes calculation concepts of the method in a feasibility study setting. Calculation examples are limited to existing without-project conditions analysis. The reader should be familiar with the material in Section 4-7, prior to studying this example.

9-2. General Study Background.

a. USACE is performing a planning feasibility study of the leveed interior area. The study area is the flood plain portion of an urban area along Smith River which encompasses 5.2 square miles and is protected from direct river flooding to the Standard Project flood protection level. The study area is heavily developed with both manufacturing and commercial businesses (Figure 9-1).

b. The interior topography slopes gently to the river. An existing 54-inch circular gravity outlet passes interior flood waters through the line-of-protection for positive head differentials.

c. The Smith River has a drainage area of approximately 10,500 square miles above the study area. Daily flow records obtained from a nearby river gage are available from 1905 to 2010.

d. Interior flooding typically occurs from moderate to heavy rainfall when the gravity outlet is blocked from high river stages. During low river stages the gravity outlet provides interior protection up to a one percent chance exceedance frequency event. Existing interior ponding is primarily limited to streets, parking lots, and a small amount of vacant land. Additional ponding locations are not economically and socially feasible.

9-3. Study Strategy.

a. General Procedure.

(1) A reconnaissance investigation has found that significant damage potential exists and that a feasibility study is justified to investigate alternative flood loss reduction plans. These plans include combinations of structural (gravity outlets, pumping facilities, and ditches) and nonstructural (flood proofing, relocation, regulations and flood warning-emergency preparedness) measures.

(2) Coincident frequency techniques are used to generate hydrologic data for flood damage evaluations, determine the optimal sizing of plan components, and define the operation criteria of

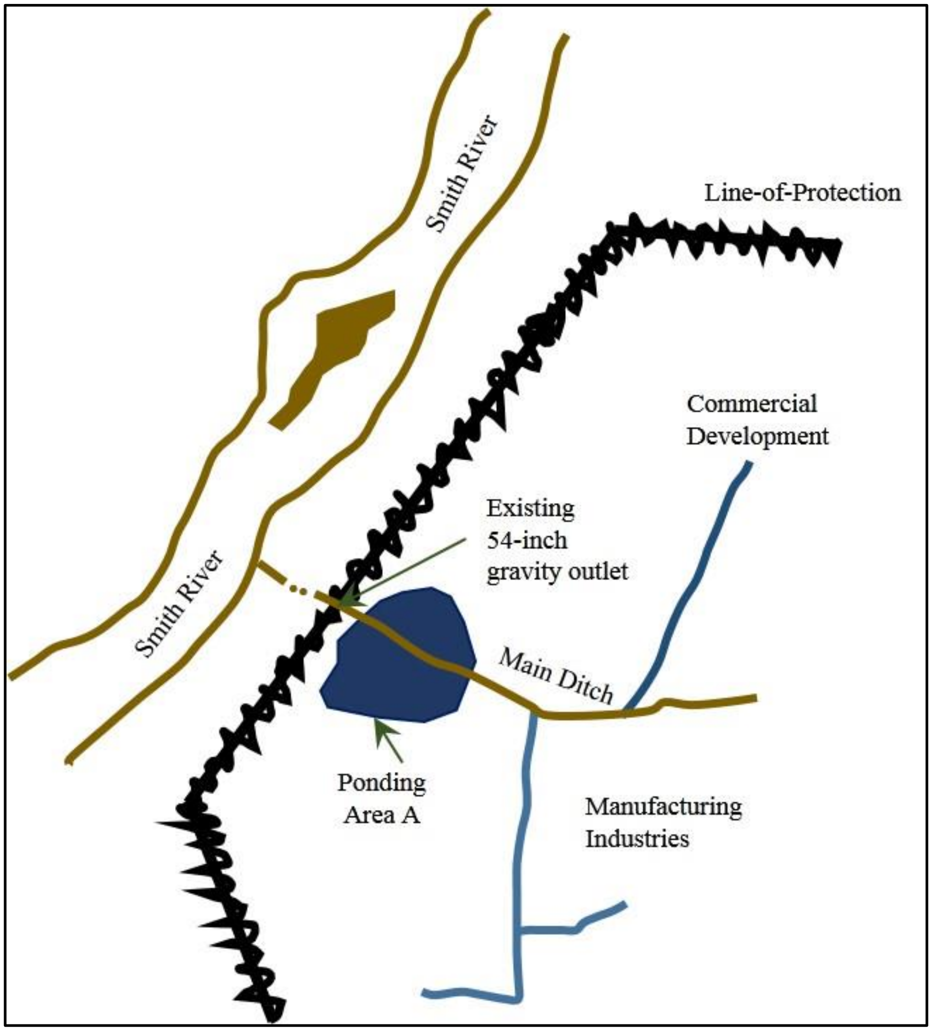

Figure 9-1. Study area map.

the adopted plan. The first step in the coincident frequency analysis is to determine the level of correlation between interior discharge and the coincident exterior stage (or discharge). For this example, observed annual maximum discharge was available for the interior area from 1972 – 2000. The correlation coefficient was computed using this annual maximum discharge dataset and the coincident (same day flow) from the Smith River daily flow time-series. Based on a computed correlation coefficient of 0.16, it was assumed that interior discharge and exterior stage could be treated as independent. Such a low correlation coefficient indicates that great interior runoff is coincident with both high and low exterior discharge, and no pattern is discernible (this is illustrated in Figure 9-2).

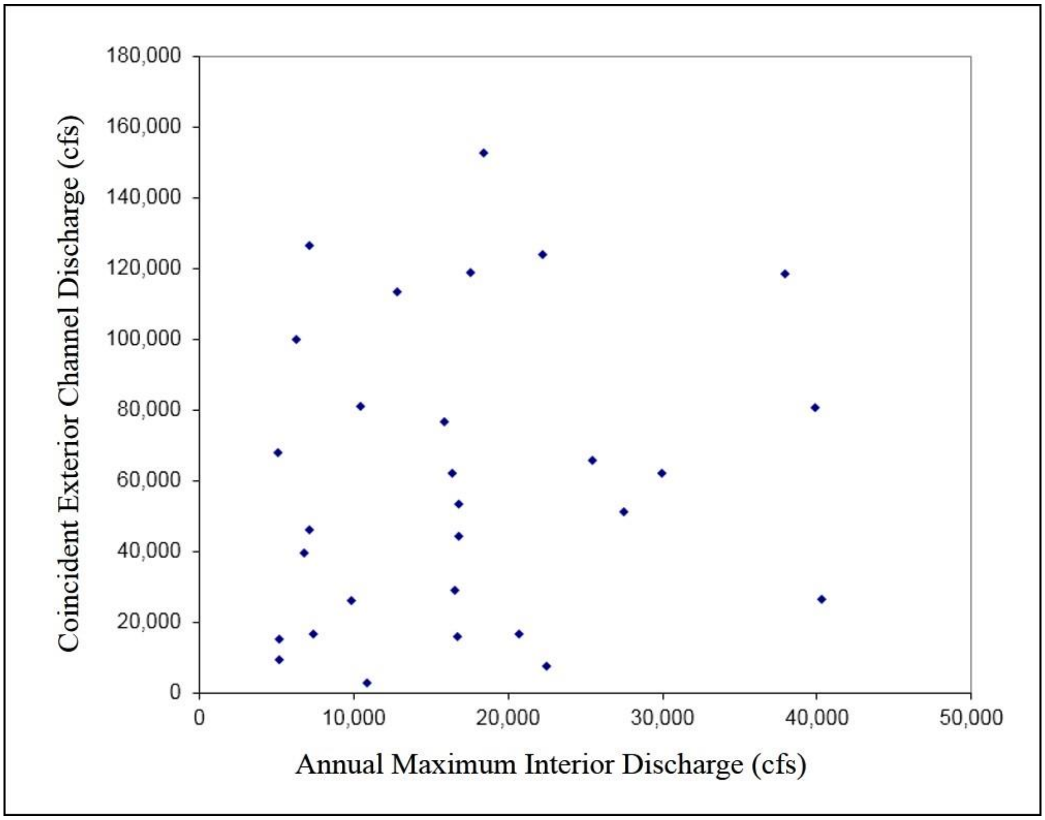

Figure 9-2. Correlation of interior and exterior flows from historic record.

(3) Adopted procedures for performing the existing condition analyses are as follows: (a) development of exterior flow data and creation of the duration curve, (b) development of the interior area flow-probability curve; using observed data and rainfall-runoff analysis of the interior area, (c) development of conditional interior stages by modeling interior stage with multiple combination of interior runoff and exterior flow, and (d) computation of the coincident interior stage-probability curve. Subsequent paragraphs detail the hydrologic analysis procedures used to develop existing conditions discharge-probability relationship.

b. Exterior Flow-Duration Analysis. Observed daily average flow for the Smith River was used to determine the flow-duration relationship, shown in Figure 9-3, at the confluence of the interior drainage and Smith River. Then the flow-duration curve was discretized into nine segments. As shown in Figure 9-3, the segments are smaller where the flow duration curve is steepest. The midpoint of each segment interval was used as the index flow value and assigned the probability for that segment. Table 9-1 shows the index locations and associated probabilities.

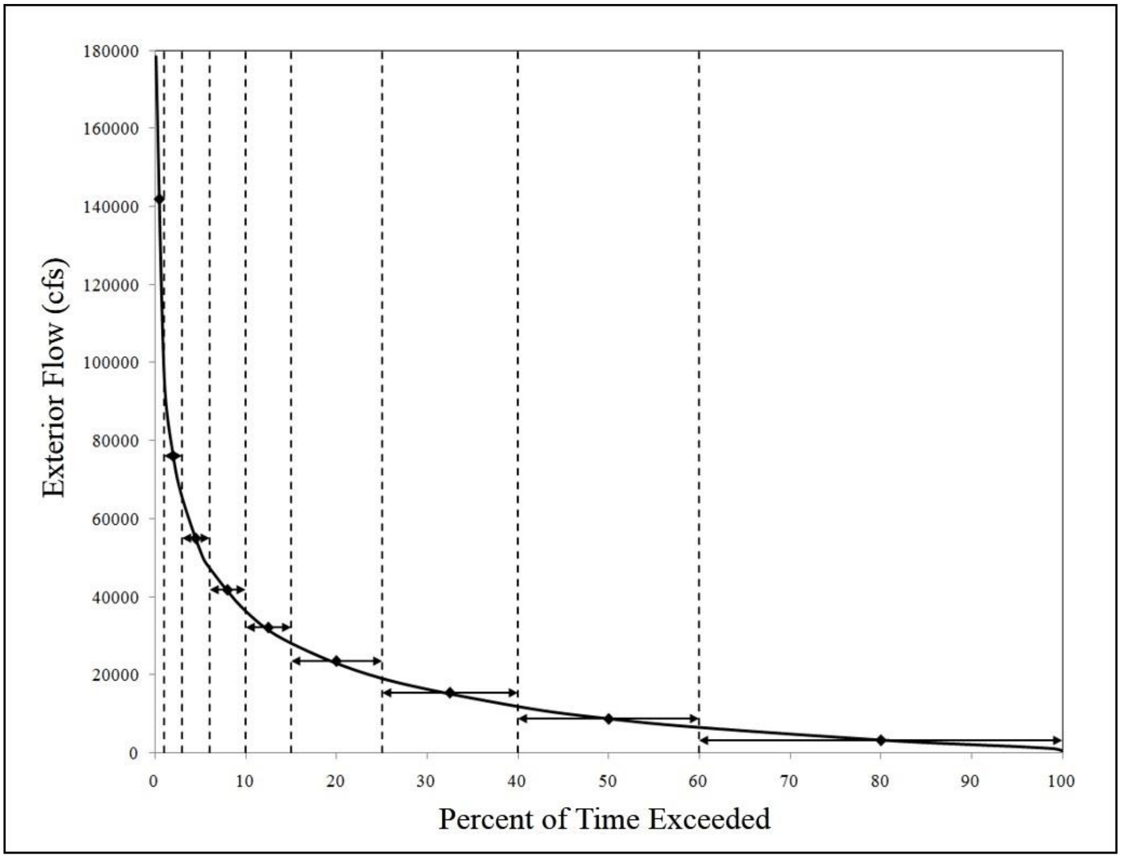

Figure 9-3. Exterior flow duration curve with index points.

Table 9-1. Index values from exterior flow-duration curve.

Flow Interval (cfs)	Index	Flow (cfs)	Probability for Segment
558 – 6550	B_9	3310	40
6550 – 11900	B_8	8780	20
11900 – 19100	B_7	15500	15
19100 – 28100	B_6	23600	10
28100 – 36300	B_5	32200	5
36300 – 47400	B_4	41850	4
47400 – 65300	B_3	55025	3
65300 – 96300	B_2	76120	2
96300 – 335000	B_1	141921	1

c. Interior Frequency Curve Analysis. Due to the fact that only twenty-nine years of record were available, the runoff-frequency curve for the interior area was developed using both observed annual peak flows and results from a rainfall-runoff model. The rainfall-runoff model was calibrated to historic events before frequency precipitation was applied. The frequency precipitation data were used to generate runoff hydrographs from the interior area. Model output from the frequency precipitation was used to define the upper, less frequent, end of the frequency curve. One-hour to ten-day precipitation data were obtained from NWS. A ten-day rainfall duration was used to generate runoff hydrographs of appropriate volume associated with the potential long periods of high river conditions. The interior area runoff-frequency curve is shown in Figure 9-4.

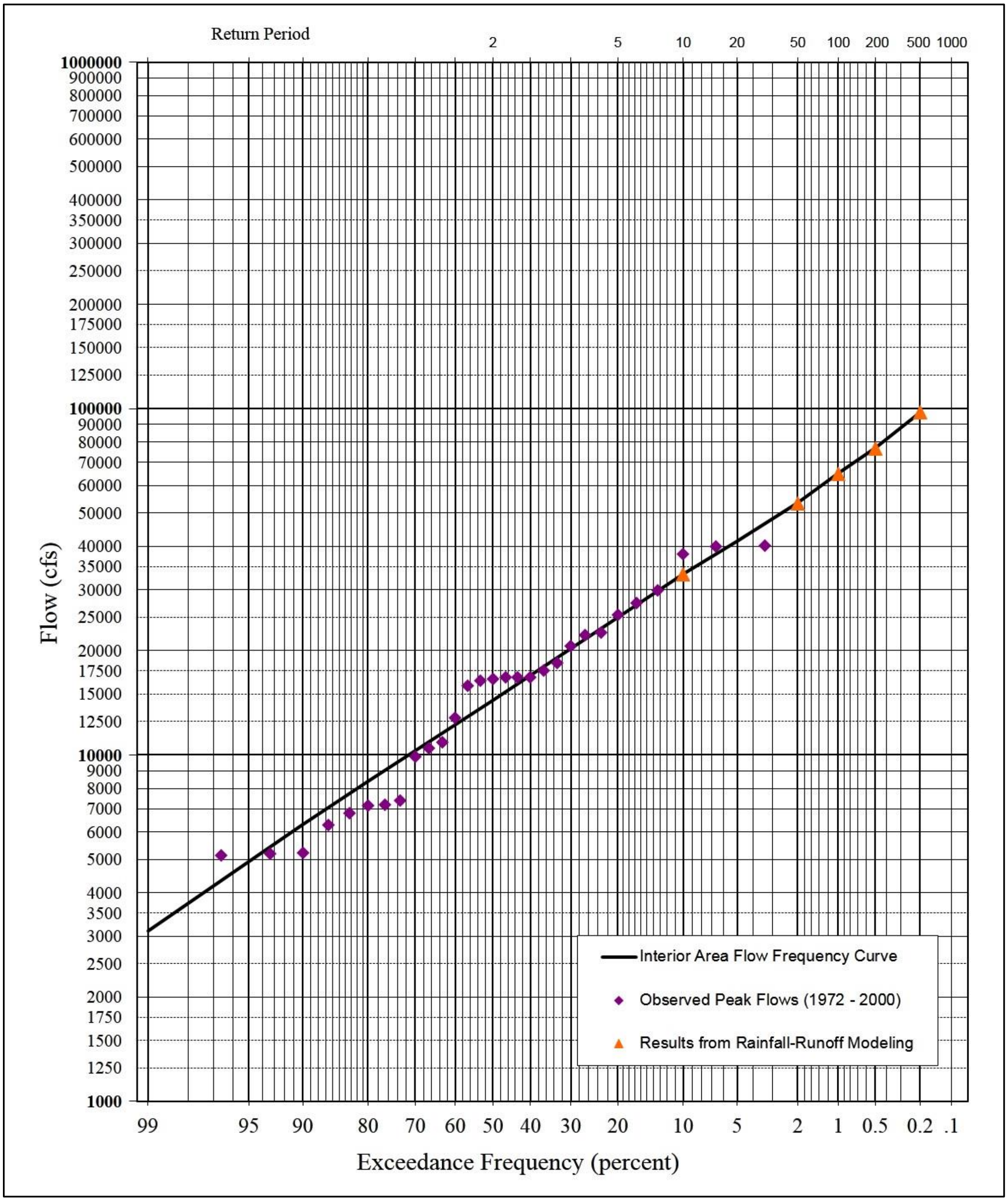

Figure 9-4. Runoff-frequency curve for the interior area.

d. Computation of Interior Stage-Probability Curve. A hydraulics model (e.g., HEC-RAS) was used to model the interior facilities. The model was used to compute the interior stage given an interior runoff value (from the interior flow-probability curve) and an exterior flow (index value from the exterior flow duration curve). As shown in Table 9-2, ninety simulations were computed that use different combinations of exterior flow and interior runoff (these are boundary conditions to the hydraulic model). For example, ten simulations were created by setting the exterior flow to 3,310 cfs, index B_9, in all ten simulations. The interior flow was set using a different value from the interior flow-probability curve. The interior flow values used corresponded to the following frequencies, 99, 95, 90, 80, 50, 10, 2, 1, 0.5, and 0.2 percent. The output from this analysis is the response of interior stage given exterior flow and interior runoff, shown in Table 9-2 and Figure 9-5. These results show that the smaller exterior flows from indexes B_9 – B_5 have little effect on the interior stage (the exterior stage must be low enough for gravity outlets to effectively drain water from interior pond). In this case, the interior stage is only a function of interior runoff. The higher exterior flows from indexes B_1 – B_4 do influence interior stage for lower interior runoff; however, results show higher exterior flows have little effect on interior stage for the largest interior runoff events. In this case, inflow into the interior pond is so large that interior stage is always greater than exterior stage.

Table 9-2. Response of interior stage given different combinations of interior and exterior flows.

Variable A Interior Flow (cfs)	**B_9 = 3,310 C = f(A,B_9)**	**B_8 = 8,780 C = f(A,B_8)**	**B_7 = 15,500 C = f(A,B_7)**	**B_6 = 23,600 C = f(A,B_6)**	**B_5 = 32,200 C = f(A,B_5)**
97,500	491.32	491.32	491.32	491.32	491.32
76,800	486.85	486.85	486.85	486.85	486.85
64,900	480.98	480.99	480.99	480.99	480.99
53,300	476.95	476.95	476.96	476.97	477.00
33,300	471.59	471.61	471.64	471.69	471.77
14,800	466.36	466.45	466.65	466.78	467.00
8,400	464.17	464.17	464.19	464.39	464.67
6,300	463.59	463.59	463.60	463.64	463.88
4,900	463.14	463.14	463.14	463.15	463.36
3,100	462.42	462.42	462.42	462.42	462.66

Variable A Interior Flow (cfs)	**B_4 = 41,850 C = f(A,B_4)**	**B_3 = 55,025 C = f(A,B_3)**	**B_2 = 76,120 C = f(A,B_2)**	**B_1 = 141,922 C = f(A,B_1)**
97,500	491.32	491.32	491.32	491.32
76,800	486.85	486.85	486.85	486.85
64,900	480.99	480.99	480.99	481.61
53,300	477.05	477.10	477.19	478.19
33,300	471.90	472.16	472.56	474.14
14,800	467.29	467.82	468.86	471.37
8,400	465.13	465.94	467.39	470.63
6,300	464.42	465.34	466.96	470.43
4,900	463.95	464.96	466.69	470.31
3,100	463.41	464.54	466.39	470.18

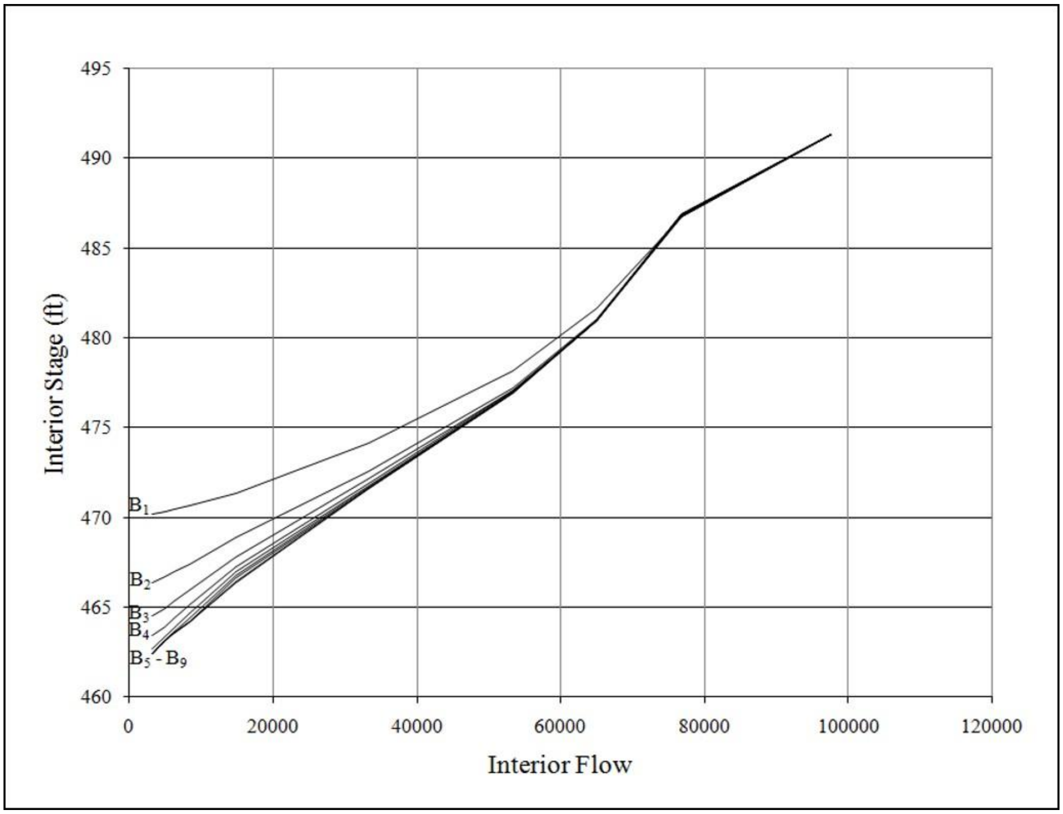

Figure 9-5. Response curves; response of interior stage given different interior and exterior flow.

e. Development of the Coincident Interior Stage-Probability Curve. Conditional interior stage-probability curves, contained in Table 9-3 and shown in Figure 9-6, were developed from the response curves by assigning the same frequency from the interior runoff to the corresponding interior stage. In this example, the interior flow values in Table 9-2 corresponded to the following probabilities; 0.002, 0.005, 0.01, 0.02, 0.10, 0.50, 0.80, 0.90, 0.95, and 0.99. The conditional probability frequency curves were subsequently used to develop a weighted stage-probability curve for the interior pond using the total probability equation. This was done by defining twenty evenly spaced interior stage values, including the minimum and maximum from the conditional-frequency curves (more stage values can be used, 20 evenly spaced values were chosen for this example). Table 9-4 contains the probability for all 20 interior stage values as extracted from the conditional interior stage-probability curves. The probabilities $P[Z_{Ci}|Z_{Bi}]$ are read from the horizontal axis in Figure 9-6. The probability from each conditional-frequency curve was multiplied by the proportion of time (probability) that was assigned to the corresponding exterior flow index. These "weighted" values from each conditional-frequency curve are then summed. Table 9-5 contains results from weighting the conditional-frequency curve results, the "Sum" column contains the total probability for each of the twenty interior stage values. Table 9-6 contains the final interior stage-probability curve, the final values were extracted from specific exceedance probabilities from the "Sum" column in Table 9-5.

Table 9-3. Conditional interior stage-probability curves.

Probability	B_9 = 3,310 $Zc_9 \vert B_9$	B_8 = 8,780 $Zc_8 \vert B_8$	B_7 = 15,500 $Zc_7 \vert B_7$	B_6 = 23,600 $Zc_6 \vert B_6$	B_5 = 32,200 $Zc_5 \vert B_5$
0.002	491.32	491.32	491.32	491.32	491.32
0.005	486.85	486.85	486.85	486.85	486.85
0.01	480.98	480.99	480.99	480.99	480.99
0.02	476.95	476.95	476.96	476.97	477.00
0.10	471.59	471.61	471.64	471.69	471.77
0.50	466.36	466.45	466.65	466.78	467.00
0.80	464.17	464.17	464.19	464.39	464.67
0.90	463.59	463.59	463.60	463.64	463.88
0.95	463.14	463.14	463.14	463.15	463.36
0.99	462.42	462.42	462.42	462.42	462.66

Probability	B_4 = 41,850 $Zc_4 \vert B_4$	B_3 = 55,025 $Zc_3 \vert B_3$	B_2 = 76,120 $Zc_2 \vert B_2$	B_1 = 141,922 $Zc_1 \vert B_1$
0.002	491.32	491.32	491.32	491.32
0.005	486.85	486.85	486.85	486.85
0.01	480.99	480.99	480.99	481.61
0.02	477.05	477.10	477.19	478.19
0.10	471.90	472.16	472.56	474.14
0.50	467.29	467.82	468.86	471.37
0.80	465.13	465.94	467.39	470.63
0.90	464.42	465.34	466.96	470.43
0.95	463.95	464.96	466.69	470.31
0.99	463.41	464.54	466.39	470.18

(1) Below is an example of how the total probability equation was used to compute the probability for an interior stage of 470.03 feet.

$$P[470.03] = P[Z_{C1}|Z_{B1}] \bullet P[Z_{B1}] + P[Z_{C2}|Z_{B2}] \bullet P[Z_{B2}] + P[Z_{C3}|Z_{B3}] \bullet P[Z_{B3}] + P[Z_{C4}|Z_{B4}] \bullet P[Z_{B4}] + P[Z_{C5}|Z_{B5}] \bullet P[Z_{B5}] + P[Z_{C6}|Z_{B6}] \bullet P[Z_{B6}] + P[Z_{C7}|Z_{B7}] \bullet P[Z_{B7}] + P[Z_{C8}|Z_{B8}] \bullet P[Z_{B8}] + P[Z_{C9}|Z_{B9}] \bullet P[Z_{B9}]$$

where:

probabilities $P[Z_{C1}|Z_{Bi}]$ are read from the horizontal axis in Figure 9-6 and
probabilities $P[Z_{B1}]$ through $P[Z_{B9}]$ are shown in Figure 9-1.

$$P[470.03] = [0.185 \bullet 0.4] + [0.187 \bullet 0.2] + [0.193 \bullet 0.15] + [0.198 \bullet 0.1] + [0.208 \bullet 0.05] + [0.224 \bullet 0.04] + [0.257 \bullet 0.03] + [0.343 \bullet 0.02] + [1.0 \bullet 0.01]$$

$$P[470.03_1] = 0.204$$

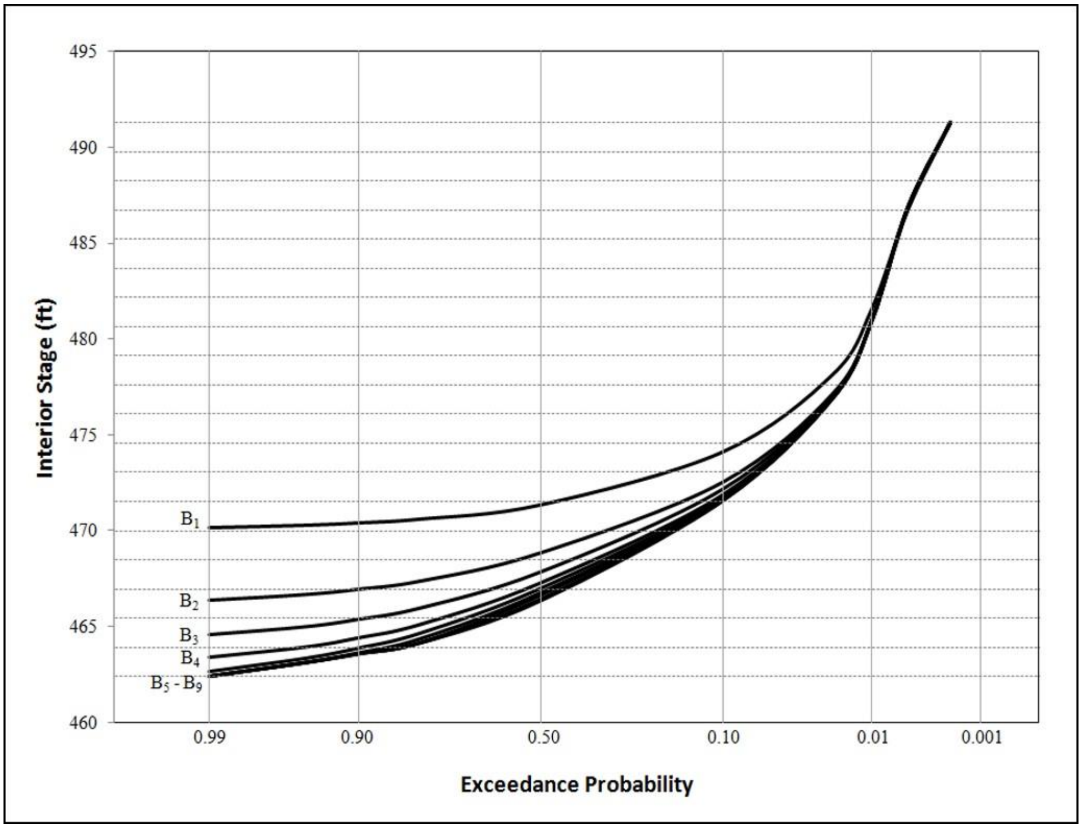

Figure 9-6. Conditional interior stage-probability curves.

Table 9-4. Probability of interior stage from each conditional-frequency curve.

Interior Stage (ft)	$P[Zc_9\|B_9]$	$P[Zc_8\|B_8]$	$P[Zc_7\|B_7]$	$P[Zc_6\|B_6]$	$P[Zc_5\|B_5]$	$P[Zc_4\|B_4]$	$P[Zc_3\|B_3]$	$P[Zc_2\|B_2]$	$P[Zc_1\|B_1]$
491.32	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
489.80	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003
488.28	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004
486.76	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005
485.24	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006
483.71	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.008
482.19	0.009	0.009	0.009	0.009	0.009	0.009	0.009	0.009	0.009
480.67	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.012
479.15	0.014	0.014	0.014	0.014	0.014	0.014	0.014	0.014	0.017
477.63	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.019	0.026
476.11	0.027	0.027	0.027	0.027	0.027	0.028	0.029	0.030	0.049
474.59	0.043	0.043	0.044	0.044	0.045	0.046	0.048	0.053	0.086
473.07	0.068	0.068	0.068	0.069	0.070	0.073	0.077	0.086	0.216
471.55	0.102	0.103	0.104	0.107	0.111	0.118	0.136	0.176	0.467
470.03	0.185	0.187	0.193	0.198	0.208	0.224	0.257	0.343	1.000
468.50	0.300	0.305	0.317	0.326	0.343	0.368	0.420	0.581	1.000
466.98	0.439	0.447	0.466	0.479	0.502	0.548	0.646	0.896	1.000
465.46	0.635	0.642	0.658	0.679	0.711	0.762	0.883	1.000	1.000
463.94	0.845	0.845	0.848	0.865	0.894	0.951	1.000	1.000	1.000
462.42	0.990	0.990	0.990	0.990	1.000	1.000	1.000	1.000	1.000

Table 9-5. Weighted probabilities.

Interior Stage (ft)	$PB_9 = 0.4$	$PB_8 = 0.2$	$PB_7 = 0.15$	$PB_6 = 0.1$	$PB_5 = 0.05$	$PB_4 = 0.04$	$PB_3 = 0.03$	$PB_2 = 0.02$	$PB_1 = 0.01$	Sum (Total Probability)
491.32	0.00080	0.00040	0.00030	0.00020	0.00010	0.00008	0.00006	0.00004	0.00002	0.002
489.80	0.00110	0.00055	0.00041	0.00028	0.00014	0.00011	0.00008	0.00005	0.00003	0.003
488.28	0.00151	0.00075	0.00056	0.00038	0.00019	0.00015	0.00011	0.00007	0.00004	0.004
486.76	0.00202	0.00101	0.00076	0.00051	0.00025	0.00020	0.00015	0.00010	0.00005	0.005
485.24	0.00243	0.00122	0.00091	0.00061	0.00030	0.00024	0.00018	0.00012	0.00006	0.006
483.71	0.00292	0.00146	0.00109	0.00073	0.00036	0.00029	0.00022	0.00014	0.00008	0.007
482.19	0.00348	0.00174	0.00131	0.00087	0.00044	0.00035	0.00026	0.00017	0.00009	0.009
480.67	0.00423	0.00212	0.00159	0.00106	0.00053	0.00042	0.00032	0.00021	0.00012	0.011
479.15	0.00552	0.00276	0.00207	0.00138	0.00069	0.00056	0.00042	0.00028	0.00017	0.014
477.63	0.00715	0.00358	0.00268	0.00179	0.00090	0.00073	0.00055	0.00037	0.00026	0.018
476.11	0.01066	0.00533	0.00402	0.00269	0.00136	0.00112	0.00086	0.00061	0.00049	0.027
474.59	0.01732	0.00869	0.00655	0.00441	0.00224	0.00184	0.00145	0.00105	0.00086	0.044
473.07	0.02701	0.01356	0.01024	0.00690	0.00352	0.00290	0.00232	0.00172	0.00216	0.070
471.55	0.04076	0.02056	0.01564	0.01067	0.00555	0.00473	0.00407	0.00352	0.00467	0.110
470.03	0.07382	0.03746	0.02895	0.01985	0.01041	0.00894	0.00772	0.00687	0.01000	0.204
468.50	0.11986	0.06099	0.04754	0.03263	0.01715	0.01471	0.01260	0.01161	0.01000	0.327
466.98	0.17573	0.08947	0.06989	0.04789	0.02512	0.02190	0.01938	0.01792	0.01000	0.477
465.46	0.25399	0.12846	0.09867	0.06787	0.03554	0.03047	0.02650	0.02000	0.01000	0.672
463.94	0.33801	0.16900	0.12718	0.08654	0.04470	0.03805	0.03000	0.02000	0.01000	0.863
462.42	0.39600	0.19800	0.14850	0.09900	0.05000	0.04000	0.03000	0.02000	0.01000	0.992

Table 9-6. Interior stage-probability curve.

Probability	**Interior Stage-Probability Curve (feet)**
0.002	491.3
0.005	486.8
0.01	481.1
0.02	477.3
0.10	471.9
0.50	466.8
0.80	464.5
0.90	463.7
0.95	463.3
0.99	462.5

9-4. <u>Summary and Discussion.</u>

a. The coincident frequency procedure described in this example is directly applicable to areas where exterior flow and interior flood events are independent. It is often useful to analyze the two extreme conditions which bracket the results prior to initiating a complete coincident frequency analysis. These conditions are (1) completely blocked gravity outlets; and (2) completely open gravity outlets. The results of these basic analyses will provide insights into whether additional studies are required, the level of detail necessary for additional studies, and identify potential alternatives to investigate.

b. Similar computation procedures are required to develop coincident stage-probability functions for existing and future with and without conditions (not presented herein). For these analyses, the interior area hydraulic model would be updated with proposed alternatives and the response curves, in Table 9-3, recomputed for each alternative.

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CHAPTER 10

Coastal Surge Overtopping Example

10-1. Overtopping Example Calculation. This example overtopping calculation is taken from the preliminary Mississippi River Gulf Outlet (MRGO) levee design east of New Orleans, LA (USACE, 2007).

a. Step 1: One percent surge elevation. The maximum one percent surge level was calculated from coupled ADCIRC-STWAVE simulations at the reach of interest. The maximum one percent surge is 15.6 feet with a maximum standard deviation of 1.2 feet.

b. Step 2: One percent wave characteristics. The maximum 1 percent wave height and the associated wave period were calculated from the same coupled ADCIRC-STWAVE simulations at the reach of interest. The maximum one percent wave parameters are H_s = 5.4 feet and T_p = 8.9 s with estimated standard deviations of 0.5 feet and 1.8 s, respectively. The bottom elevation 600 feet from the shoreline is approximately zero feet NAVD88 2004.65. The one percent surge elevation is 15.6 feet, so the one percent wave height is about 35 percent of the water depth. This implies that the foreshore can be considered as shallow (H/h < 1/3) and breaking will be very limited towards the toe of the levee.

c. Step 3: Mean overtopping rate. The proposed cross-sectional profile is given in Figure 10-1. The mean overtopping rate is calculated from Equation VI-5-24 of EM 110-2-1100 for levees:

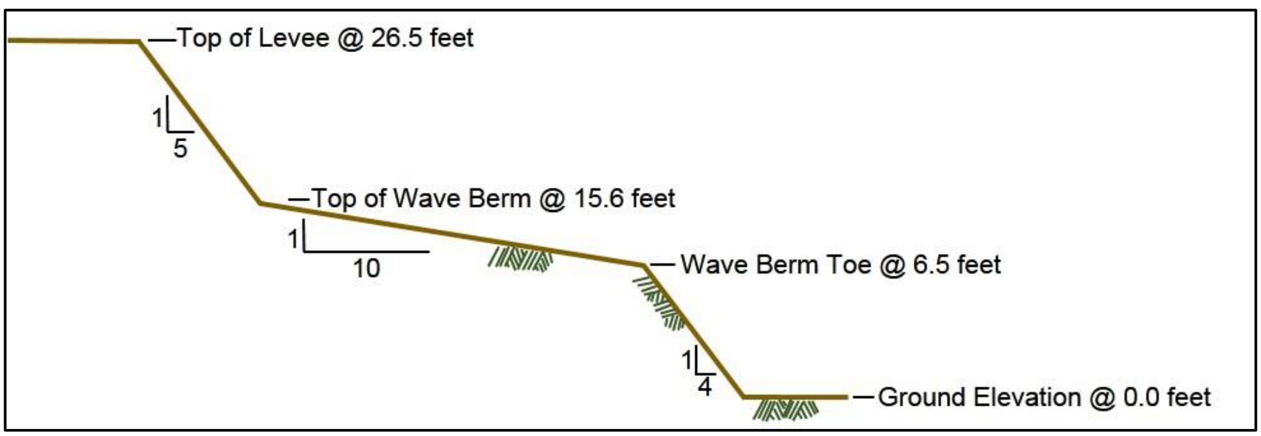

Figure 10-1. Preliminary levee cross-section design (USACE, 2007).

where:

H_s = 5.4 ft, T_p = 8.9 sec, slope of the upper levee is 1:5 (tan α = 1/5)
s_{op} = $(2\pi/g)(H_s/T_p^2)$ = 0.0133

ξ_{op} = tan α/(s_{op})$^{1/2}$ = 1.73 < 2, to select between Equations VI-5-24 and VI-5-25 (EM 1110-2-1100, Part VI, Table VI-5-11, page VI-5-32). Equation VI-5-24 is applicable in this example:

$$\frac{q}{\sqrt{gH_s^3}}\sqrt{\frac{s_{op}}{tan\alpha}} = 0.06 \exp\left(-5.2\frac{R_c}{H_s}\frac{\sqrt{s_{op}}}{tan\alpha}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta}\right)$$

R_c is the freeboard (levee height minus the 1 percent water level) equal to 26.5 feet minus 15.6 feet equals 10.9 feet,

γ coefficients represent effects of slope roughness, berm, shallow water and wave direction, and guidance for selecting the values given in EM 1110-2-1100.

For this example,

$\gamma_r = 1$,
$\gamma_b = 0.77$,
$\gamma_h = 1$, and
$\gamma_\beta = 1$

For this cross section, the mean overtopping rate is q = 0.006 cfs/feet of levee.

d. Step 4: Uncertainties. The uncertainty analysis is carried out to determine the fifty percent and 90 percent exceedance overtopping rates for the one percent event. The analysis is based on a Monte Carlo simulation of the overtopping rate with the water level and wave inputs selected randomly from a normal distribution using the means and standard deviations for the one percent water level, wave parameters, and coefficient in Equation VI-5-24 of EM 1110-2-1100 equation ($\sigma = 0.55$ for the coefficient 5.2). The result of the uncertainty analysis is shown Rin Figure 10.2. Figure 10-2 shows the frequency curve of the overtopping rate given the mean values and standard deviations of the one percent water level (15.6 feet/1.2 feet), the wave height (5.4feet/0.5feet) and the wave period (8.9seconds/1.8seconds). The overtopping rate at the upper 90 percent confidence limit is 0.06 cfs/feet, and the best estimate overtopping rate equals 0.005 cfs/feet. Both overtopping rates show that this cross-section meets the design criteria ($q_{90} < 0.1$ cfs/feet and $q_{50} < 0.01$ cfs/feet).

e. Step 5: Resilience for events above design level. Resilience is investigated using the 0.2 percent values for the hydraulic boundary conditions. For this example, the 0.2 percent surge is 19.9 feet, the significant wave height is 8.0 feet, and the peak period is 14.4 s. The exceedance frequency curve of the 0.2 percent overtopping rate was computed with the 1 percent design values and the 0.2 percent hydraulic boundary conditions. The results are shown in Figure 10.3. The 50 percent exceedance overtopping rate is approximately 2 cfs/feet, which is about 200 times greater than the one percent design criterion (0.01 cfs/feet). This may indicate that the chance of survival of this design during a 0.2 percent event is low.

10-2. Extratropical Storm Analysis. Extratropical storms generally occur with a much greater frequency than hurricanes. Extratropical events cannot be parameterized in the same way as hurricanes; therefore, the development of stage-probability relationships based on the JP< is not a viable approach for extratropical events. Three approaches are commonly used for developing site-specific frequency relationships. These are based on: historical data, synthetic data, and the Empirical Simulation Technique (EST).

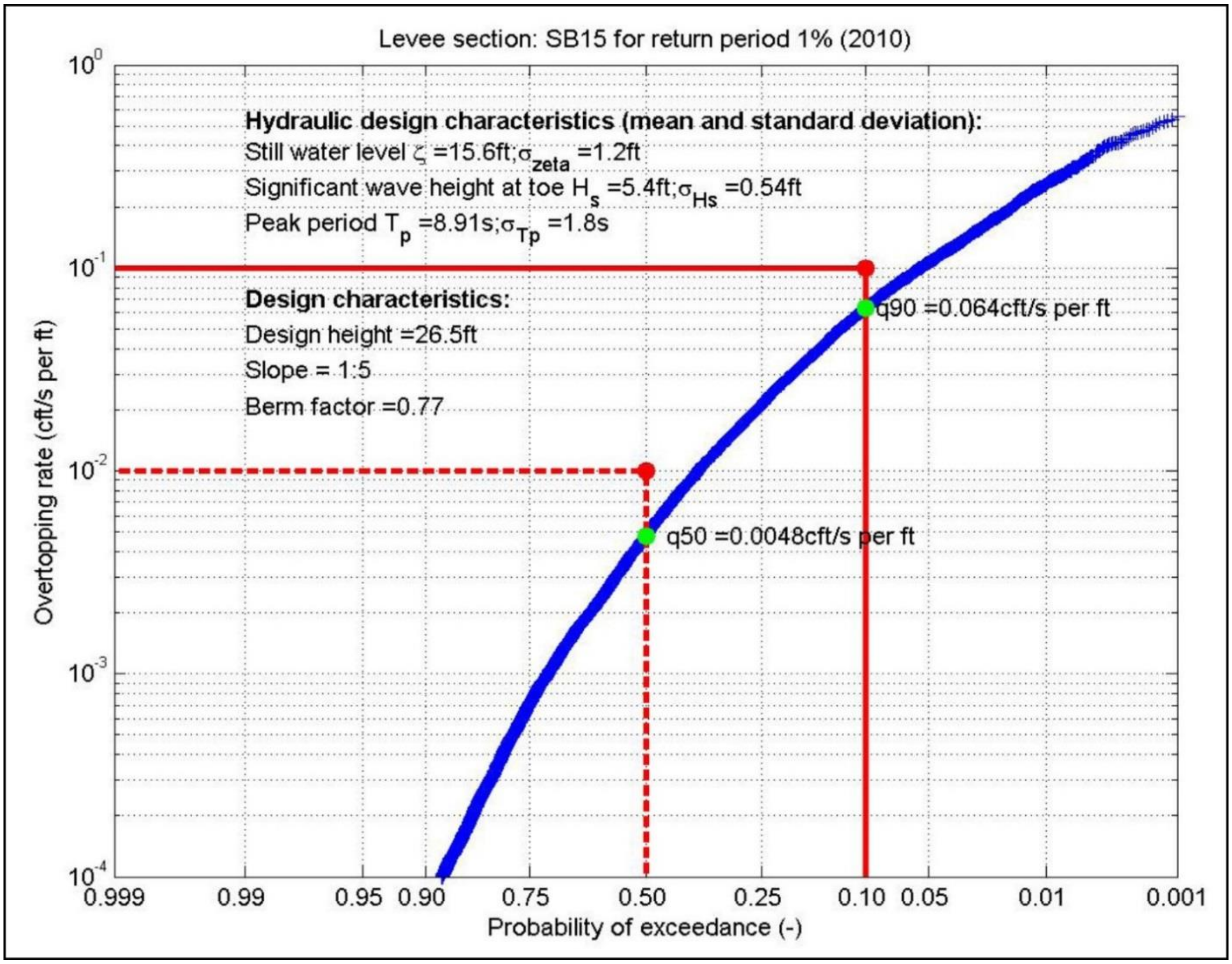

Figure 10-2. Overtopping rate as a function of the probability of exceedence for the MRGO Levee for the 1% event (USACE, 2007).

a. Historical Approach. The historical approach requires a database of historical storm surge measurements for the study area and requires that these measurements provide a representative sample of all possible events for the site. Many coastal locations do not have adequate historical data to develop frequency-of-occurrence relationships. Even if many years of data were available, there is no assurance that the data represents the full population of possible storm events. A Peak Over Threshold (POT) method is applied to identify the peak water levels for each event exceeding a given threshold. The threshold is selected to identify on average one to ten events per year. Tropical events must be excluded. A CDF is constructed by plotting event peak water level against probability of exceedance, based on ranking the events. Typically, the period of record is less than required return period of analysis (e.g., fifty-year return period water levels is required and only thirty years of data were available), so the historic data were fit to an extremal distribution, such as Generalize Extreme Value, Fisher-Tippett, or Log Pearson Type III distribution, to extrapolate the probability density function.

b. Synthetic Approach. If sufficient measured water level data were not available at the project site, synthetic data can be generated through numerical modeling. Extratropical storms are identified using nearby water level measurements or other storm measures (e.g., peak wind speeds). Water level modeling requires input wind fields, which may be pulled from the twenty- to thirty-year database of hindcast storms of WIS along the coasts of the United States. Storm wind fields, tides and wave stresses are input to a numerical hydrodynamic long-wave model to produce storm surge (Figure 5-2). The same POT method used with historic data were applied to identify the storm peaks, and the peak water levels are ranked and plotted to generate CDFs fit to external distributions.

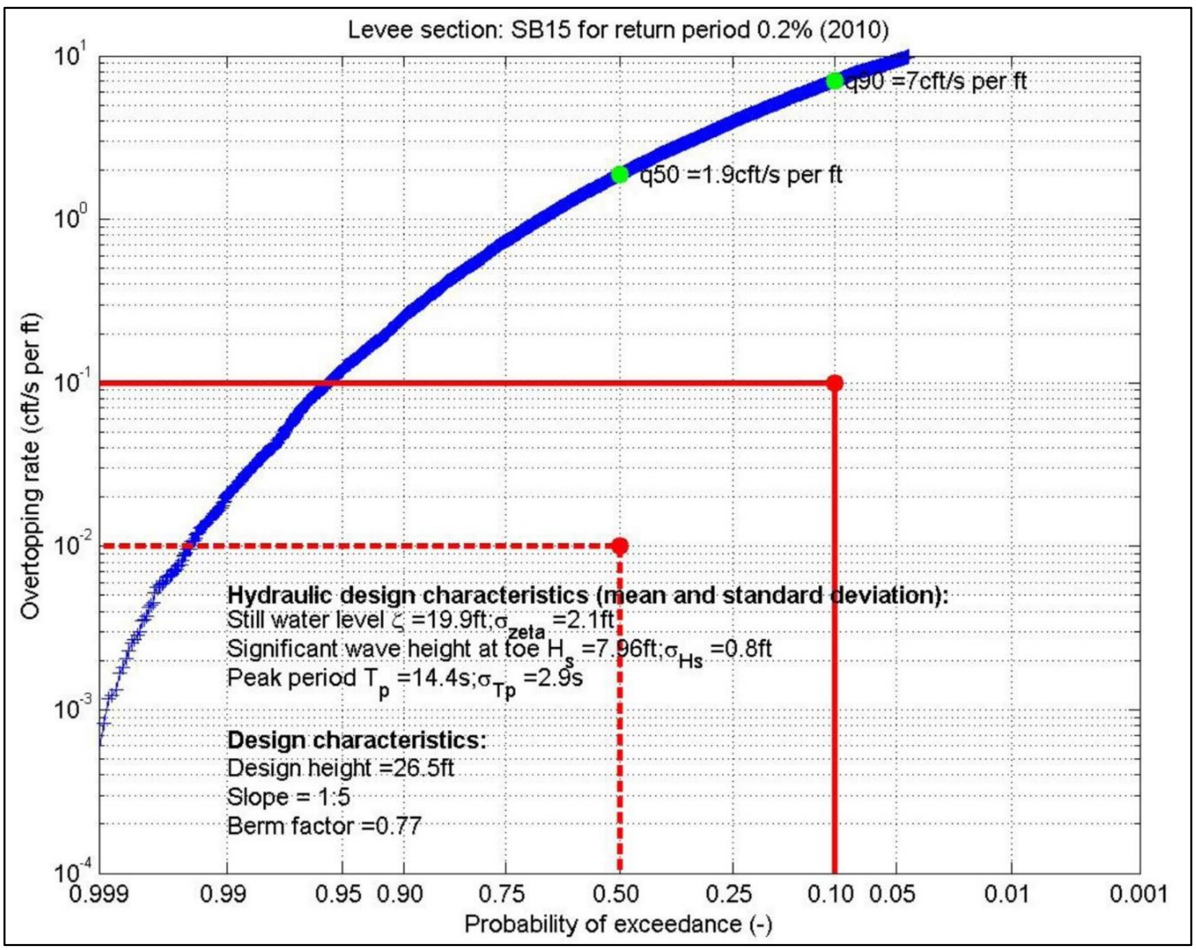

Figure 10-3. Overtopping rate as a function of the probability of exceedance for the MRGO Levee for the 0.2% event (USACE 2007).

c. Empirical Simulation Technique (EST). The recommended method for performing extratropical water level frequency analysis is the EST (Scheffner, 1999). The EST employs a Monte Carlo simulation approach that includes multiple life cycle simulations of water level (or other storm response parameters) based on random sampling of the historic storm climate, but also adds variations based on a random walk interpolation scheme based on nearest neighbor storm response. EST assumes that future events will be similar to past events. The method does not rely on assumed parametric relationships, but uses the joint probability relationships inherent in the database, and thus avoids unrealistic (non-physical) events. In the EST, a training set of storms is selected from historic data or simulations using POT. The tide signal should be removed from measured water levels by subtracting predicted tides. This procedure produces residual water levels which reflect atmospheric and wave forcing. In this way the storm selections are based on atmospheric forcing rather than tidal considerations. Selection of the training set is critical and must define the storm climate because it serves as the basis for defining the future events. The EST requires an input vector that characterizes the storm it represents (in the simplest application, this is the measured or simulated peak storm water level) and a response vector (which is the water level at the site of interest) for all storms in the training set. Additional inputs include the length of simulation (to calculate water levels with a 1 percent chance of occurrence, a 100-year simulation is required), the number of realizations to simulate (typically 300 - 1,000), and weighting for each storm (typically they are equally weighted). The number of storms per year is characterized by a Poisson distribution. The EST output is the mean water level CDF over all the realizations and confidence bands. EST software is included in the Corps Coastal Design and Analysis System (http://chl.erdc.usace.army.mil/cedas).

d. Tides. The typical duration of extratropical storms exceeds the duration of the tidal cycle (12.5 to 25 hours) and the storm surge hydrograph of extratropical storms is typically broad. Therefore, it can be assumed that high tide will coincide with peak storm surge. This assumption eliminates the need to consider tide phase and aligning peak surge at various phases of the tide. However, consideration of the spring-neap tidal cycle is required. A simple method to include the peak tidal variation is determine the mean of the highest 25 percent (spring), middle fifty percent (mean), and lowest 25 percent (neap) tidal amplitudes from the predicted tides over the 19-year tidal epoch. Then expand the EST training set of storms by a factor of three by linearly adding the spring, mean, and neap tidal amplitudes to the water level (or storms could be simulated at mean high tide with the increments to spring and neap tide added). Instead of equal weight to all storms, now the storms with the mean tidal amplitude added are double weighted. If water levels are strongly nonlinear with tide, simulations at the three water levels may be required (at significantly larger computational effort).

e. Waves and Overtopping. Wave height and associated period CDFs can be calculated using the same methods used for water level (historical, synthetic, or EST). Once the CDFs for water level and waves are computed, overtopping rates can be calculated using the same method described for extratropical storms.

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APPENDIX A

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Real Estate Handbook.

ER 1105-2-100
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ER 1105-2-101
Risk Analysis for Flood Damage Reduction Studies.
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ER 1110-2-1150
Engineering and Design for Civil Works Projects.
https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-2-1150.pdf

ER 1110-2-1302
Civil Works Cost Engineering.
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ER 1100-2-8162
Incorporating Sea Level Changes in Civil Works Programs
https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1100-2-8162.pdf

EP 1110-2-9
Hydrologic Engineering Studies Design
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GLOSSARY

Terms and Abbreviations

This glossary consists of an explanation of terms and abbreviations, which includes symbols for this manual and a dictionary of additional terms related to risk and uncertainty analysis.

Glossary-1. Terms.

Agricultural Areas: Lands intended primarily for crop production, pastures, and other similar uses; including the closely associated facilities of on-farm roads, fences, etc.

Base Conditions: The land use and related conditions expected to exist at the beginning of the first year of project operation.

Base Year: The year the proposed project is expected to be operational.

Blocked Gravity Conditions: Conditions that exist when exterior stages are higher than interior stages, thus, preventing flow of interior flood waters through the gravity outlets.

Coincident Probability: Probability of flooding, exceeding a given elevation based on the probability of flooding from each source of flooding.

Conditional Probability: The probability of flooding from one source given the condition of flooding from another source.

Correlation: The degree to which flooding from one source occurs or can be predicted from flooding from another source.

Dependence: The degree to which flooding of an area from one source is related to (usually in a physical sense) flooding from another source.

Detention Storage Area: Any low area near the inlets to gravity outlets, pumping stations, or pressure conduits used to temporarily store interior flood waters in excess of the rate at which these flows can be passed through the line-of-protection.

Discrete Events: Flood events in a series which may be considered individually since they are independent of other flood events in the series.

Diversions: Ditches/conduits designed to bypass flood waters around/away from a specific area.

Existing Conditions: The present land use and related conditions occurring under existing and authorized improvements, laws, and policies.

Exterior Stage: Water surface level on the unprotected (exterior) side of the line-of-protection.

Future Conditions: The most likely land use and related conditions expected in the future. Other conditions than those deemed the most likely may also be considered future considerations.

Gravity Outlets: Culverts, conduits, or other similar conveyance openings through the line-of-protection that permit discharge of interior floodwaters through the line-of-protection by gravity when the exterior stages are relatively low. Gravity outlets are equipped with gates to prevent river flows from entering the protected area during time of high exterior stages.

Independence: Flooding of an area from one source is unrelated to flooding from another source.

Interception Systems: Sewers/ditches provided to connect existing sewers of channels discharge through line-of-protection by means of gravity outlets, pumping stations, or pressure conduits.

Interior Stage: Water surface level on the protected side of the line-of-protection.

Interior System: Structural/nonstructural flood loss reduction measures located behind line-of-protection. These measures may consist of water management measures of gravity outlets, pumping stations, interior detention storage, diversions, pressure conduits, hillside reservoirs, and facility protection measures of flood proofing, structure relocation, and development management measures of floodplain regulations, and flood emergency warning/preparedness planning measures.

Line-of-Protection: Location of levee or wall that prevents floodwaters from entering an area.

National Economic Development (NED) Plan: The plan which maximizes net national economic development benefits.

Nonstructural Measures: Measures designed to reduce flood losses by implementation of facility flood proofing, raising, or relocation; and development of regulations and flood warning/ emergency preparedness planning actions.

Pressure Conduits: Closed conduits designed to convey interior flows through the line-of-protection under internal pressure. The inlet to a pressure conduit that discharges interior flows by force of gravity must be at a higher elevation than the river stage against which it functions. Some pressure conduits may serve as discharge conduits from pumping stations.

Pumping Station: Pumps located at or near the line-of-protection to discharge interior flows over or through the levees or flood walls (or through pressure lines) when free outflow through gravity outlets is prevented by high exterior stages.

Residual Damage: Flood damage remaining after implementation of flood loss reduction measures.

Structural Measures: Measures designed to reduce flood losses by construction of levees, gravity outlets, pumping stations, detention storage, reservoirs, and diversions.

Tie Back Levee: A levee that extends from the river, lake, or coast to a bluff line and is part of the line-of-protection.

Urban Area: Area presently or expected to be developed for residential, commercial, or industrial purposes within the period considered in project formulation.

Glossary-2. Abbreviations.

ADCIRC	Advanced CIRCulation Model
AEP	annual exceedance probability
CDF	cumulative distribution function
cfs	cubic feet per second
CNP	conditional non-exceedance probability
COULWAVE	Cornell University Long & (Intermediate) Wave modeling package
DDR	Design Documentation Report
EAD	expected annual damage
EC	Engineering Circular
EM	Engineering Manual
ER	Engineering Regulation
ERDC	Engineer and Research Development Center
EST	Empirical Simulation Technique
GSSHA	Gridded Surface/Subsurface Hydrologic Analysis (software)
HEC	U.S. Army Corps of Engineers, Hydrologic Engineering Center
HEC-FDA	Flood Damage Reduction Analysis (software)

HEC-HMS	Hydrologic Modeling System (software)
HEC-RAS	River Analysis System (software)
HEC-SSP	Statistical Software Package (software)
HQUSACE	Headquarters, U.S. Army Corps of Engineers
IPET	Interagency Performance Task Force
IWR	Institute for Water Resources
JPM	Joint Probability Method
JPM-OS	Joint Probability Method with Optimal Sampling
km	kilometer
mb	millibar
mm	millimeter
MRGO	Mississippi River Gulf Outlet
NM	nautical mile
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Survey
NRCS	Natural Resource Conservation Service
NWS	National Weather Service
PBL	Planetary Boundary Layer
POT	Peak Over Threshold method
SMART	Specific Measurable Attainable Risk Informed Timely Planning Principles
SSURGO	Soil Survey Geographic database
STWAVE	Steady-state WAVE model
SWAN	Simulating Waves Nearshore model
USACE	U.S. Army Corps of Engineers

USGS	U.S. Geological Survey
WAM	Wave Predication Model
WIS	Wave Information Studies (USACE)

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