# 7

# Designing a Levee

## Key Messages

This chapter will enable the reader to:

- **Understand the best and evolving practices.** Design practices for levees continue to evolve based on a better understanding of levee performance.
- **Scale the design to the problem.** The level of investigation and study should be risk-informed and scaled to the size and nature of the work.
- **Adapt to specific design features.** Each levee feature requires specific design considerations appropriate to its function, form, and failure modes.
- **Incorporate resilience features.** Add features that absorb adverse loadings without breaching, facilitate rapid recovery, and provide the potential for strengthening and adapting to changing hazards and consequences.


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Other chapters within the National Levee Safety Guidelines contain more detailed information on certain topics that have an impact on designing a levee, as shown in Figure 7-1. Elements of those chapters were considered and referenced in the development of this chapter and should be referred to for additional content.

### Figure 7-1: Related Chapter Content

| CH 1 | CH 2 | CH 3 | CH 4 |
|---|---|---|---|
| Levee form and function<br>Types of levee projects | Levee features | Engaging for levee projects | Flood and levee risk<br>Risk assessment |
| **CH 5** | **CH 6** | **CH 7** | **CH 8** |
| | Levee alignment<br>Crown elevation and geometry | **Designing a Levee** | Instrumentation<br>Construction and utility considerations |
| **CH 9** | **CH 10** | **CH 11** | **CH 12** |
| Instrumentation and monitoring | | | Community resilience |

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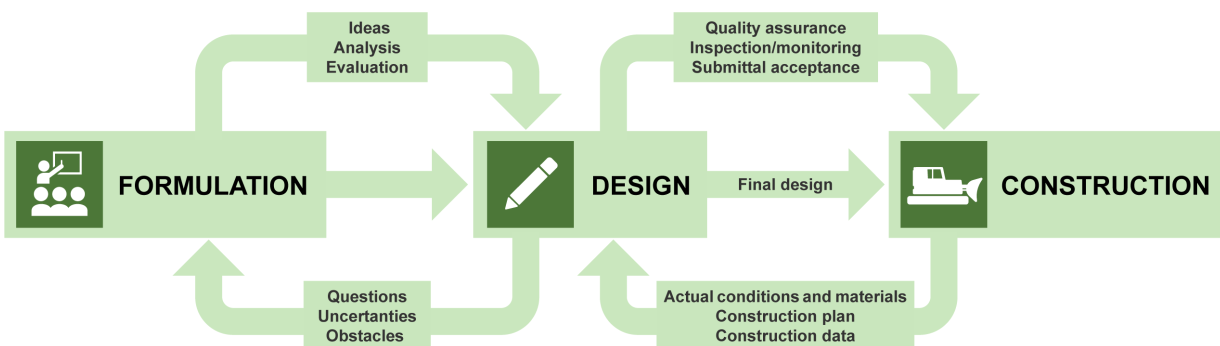
# 1 Introduction

The purpose of this chapter is to present best practices, criteria, and design principles for levee features that reduce risk posed from coastal, riverine, or rainfall flooding. As described in **Chapter 2**, levee features include earthen embankments, floodwalls, seepage control systems, closure structures, transitions between features, interior drainage systems, and instrumentation for construction and post-construction operation and monitoring of the levee. Coordinating design with levee formulation, risk assessment, and construction activities is also addressed.

The guidance in this chapter is provided for use by qualified professional engineers, planners, and floodplain managers working with federal, state, and local regulators; levee owners; and contractors specializing in levee formulation and design.

The guidelines provided in this chapter should be read in conjunction with the those in **Chapters 6 and 8** because the three processes have close connection, as illustrated in Figure 7-2. In addition, the guidelines in this chapter should be applied to conceptual, feasibility, and final design phases of a levee project, whether the project is a new levee or an existing levee requiring modification or rehabilitation.

**Figure 7-2: Interaction Between Formulation, Design, and Construction**



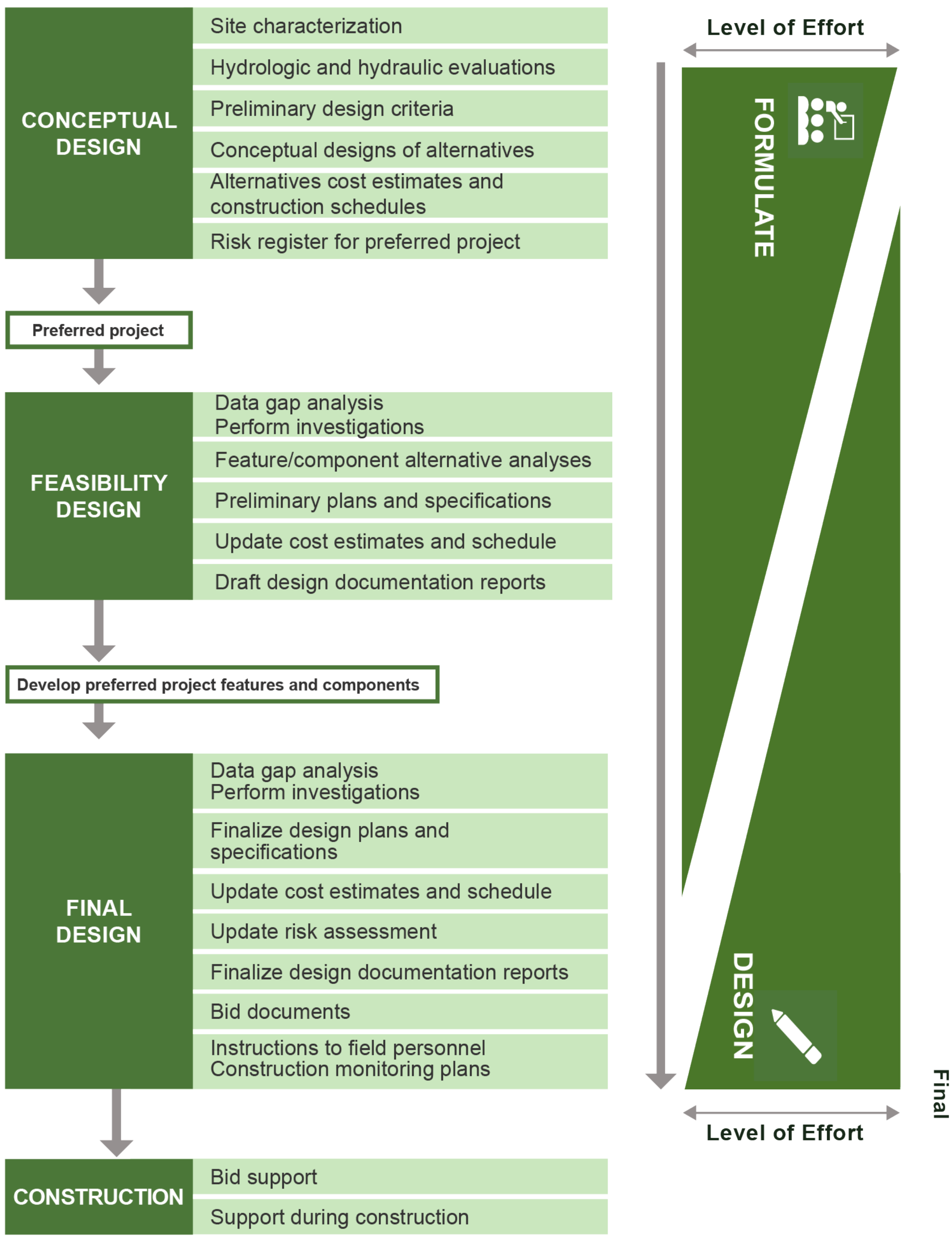

# 2 Design Process, Principles, and Considerations

## 2.1 Design Process

Design is an iterative process, involving multiple steps that may be repeated in each phase that progressively increases the level of design detail. Figure 7-3 shows the typical process for general engineering design. Details are provided in subsequent sections of this chapter describing the variations in design practices for levee and floodwall features.

Formulation of a levee project starts with the realization that there is a need for action (**Chapter 6**). From there, an idea or solution is developed. This idea gets expanded and refined throughout the formulation process. As the idea or solution becomes more defined, there is enough information to start the design process. Levee formulation and design evolve in parallel, as shown on the right-hand side of Figure 7-3, and each informs the other during this evolution. Over time, the level of effort for formulation recedes and that of design increases until the final design is reached.

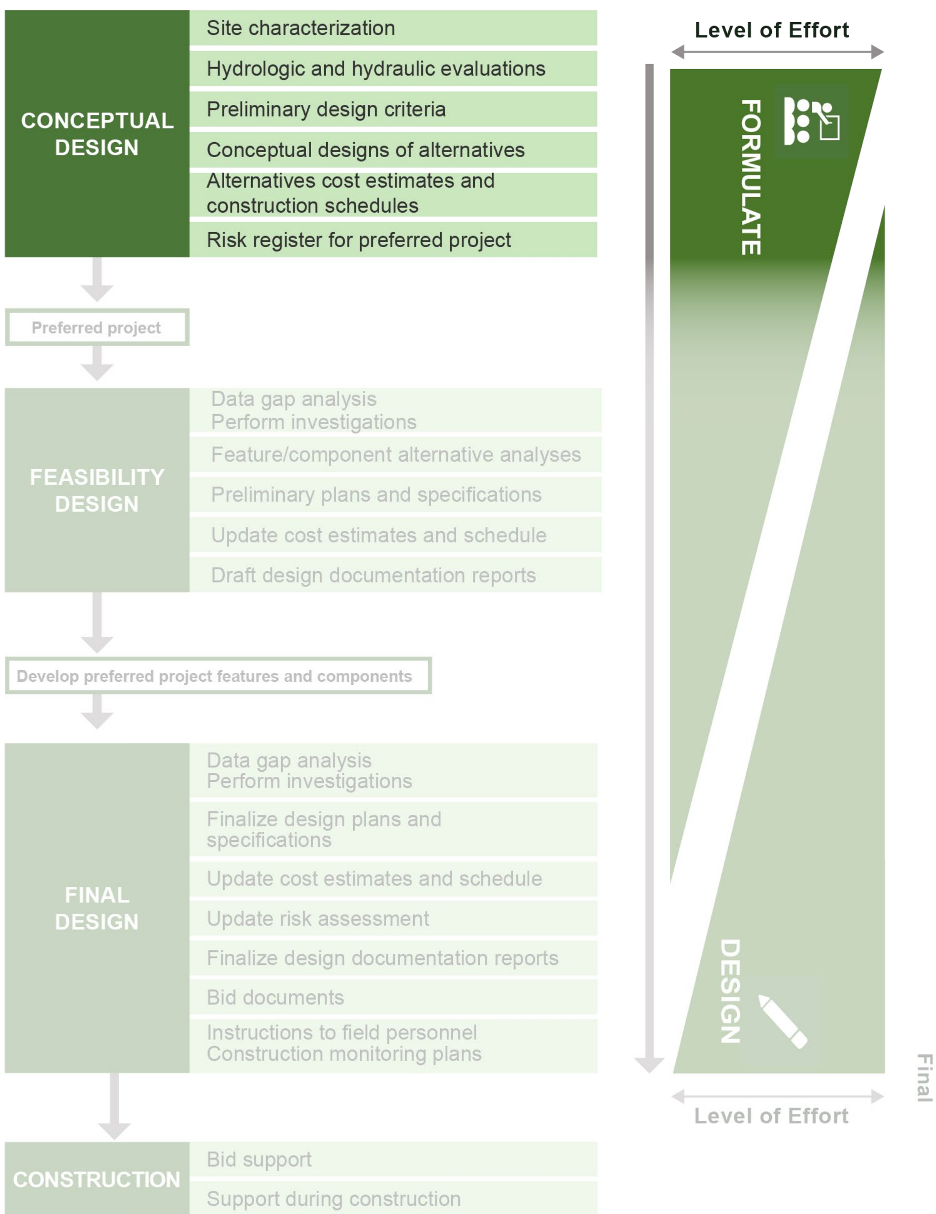
### Figure 7-3: Design Process



### 2.1.1 Conceptual Design Phase

The conceptual design phase supports the project formulation process (**Chapter 6**) in a high-level evaluation of alternatives to identify a preferred project alternative that meets the project objectives (Figure 7-4). Project formulation activities such as potential failure mode analyses, risk assessments (**Chapter 4**), and community engagement (**Chapter 3**) are key sources of information to guide the conceptual design phase especially when evaluating alternatives. The formulation process should inform the design process about important constraints and opportunities for the conceptual designs (e.g., environmental, cultural, and political). Determination of constraints, opportunities and site characterization are particularly important for modification/rehabilitation projects, especially when the projects are in more urban areas where significant development has taken place since the original levees were constructed.

**Figure 7-4: Process for Conceptual Design**



Conceptual design phase for new levees and levee modification/rehabilitation projects (Figure 7-4) typically include:

- Performing site characterization (data gathering) for topography, geotechnical/geological conditions, existing infrastructure and utilities, and real estate boundaries (section 3). Note the following:
  - After reviewing available data and visiting the alternative levee alignments, the design team may recommend performing some limited site investigations (e.g., topographic mapping, geotechnical investigations, utilities) to better characterize site conditions in critical areas.
  - During the design of a levee modification or rehabilitation, utilities that were installed after construction of the original levee should be characterized and evaluated to establish if they should be remediated or relocated for levee safety or to facilitate construction of the levee. It is not uncommon to include utility modifications or relocations as part of levee modification or rehabilitation projects to reduce or eliminate utility risks to the levee system and to comply with applicable federal, state, and local regulations. Refer to **Chapter 8** for utility considerations for levee construction.
- Completing hydrologic and hydraulic evaluations (including any associated coastal storm surge and/or riverine hydraulic studies), taking into account changing conditions like those associated with climate change. Refer to **Chapter 6** for more information on hydrologic and hydraulic evaluations. When performing these evaluations, the following should be performed:
  - Evaluation results should be combined with other factors (e.g., wave action, potential levee settlement, overbuild, and resilience considerations) to establish the minimum top-of-levee elevations (and cross sections of coastal levees) to meet flood risk management objectives.
  - If managing interior drainage is a significant factor when evaluating alternative design concepts, then interior drainage studies should be considered in this phase.
  - For an existing levee project, current coastal storm surge, riverine flooding, and interior drainage studies may require updating for levee modification and rehabilitation projects, particularly if climate change impacts were not previously considered.
- Developing appropriate design criteria (section 2.2.1) for the conceptual designs.
- Preparing conceptual designs for each alternative project, including levee alignments, top of levee elevations, and associated features to a level of detail sufficient to define the alternative. Guidance on selection of levee alignments, determining design water levels and level crest levels can be found in **Chapter 6**.
- Developing the preliminary cost estimates (section 2.3.2), construction schedules (section 2.3.4), and drafting the risk register for the preferred project. See **Chapter 8** for more information on risk registers.

On completion of the conceptual phase, the preferred project (including features) should be optimized to achieve project goals as part of the project formulation process. A similar process is achieved in the feasibility and final design phases through more in-depth studies, usually after more site characterization data has been collected.

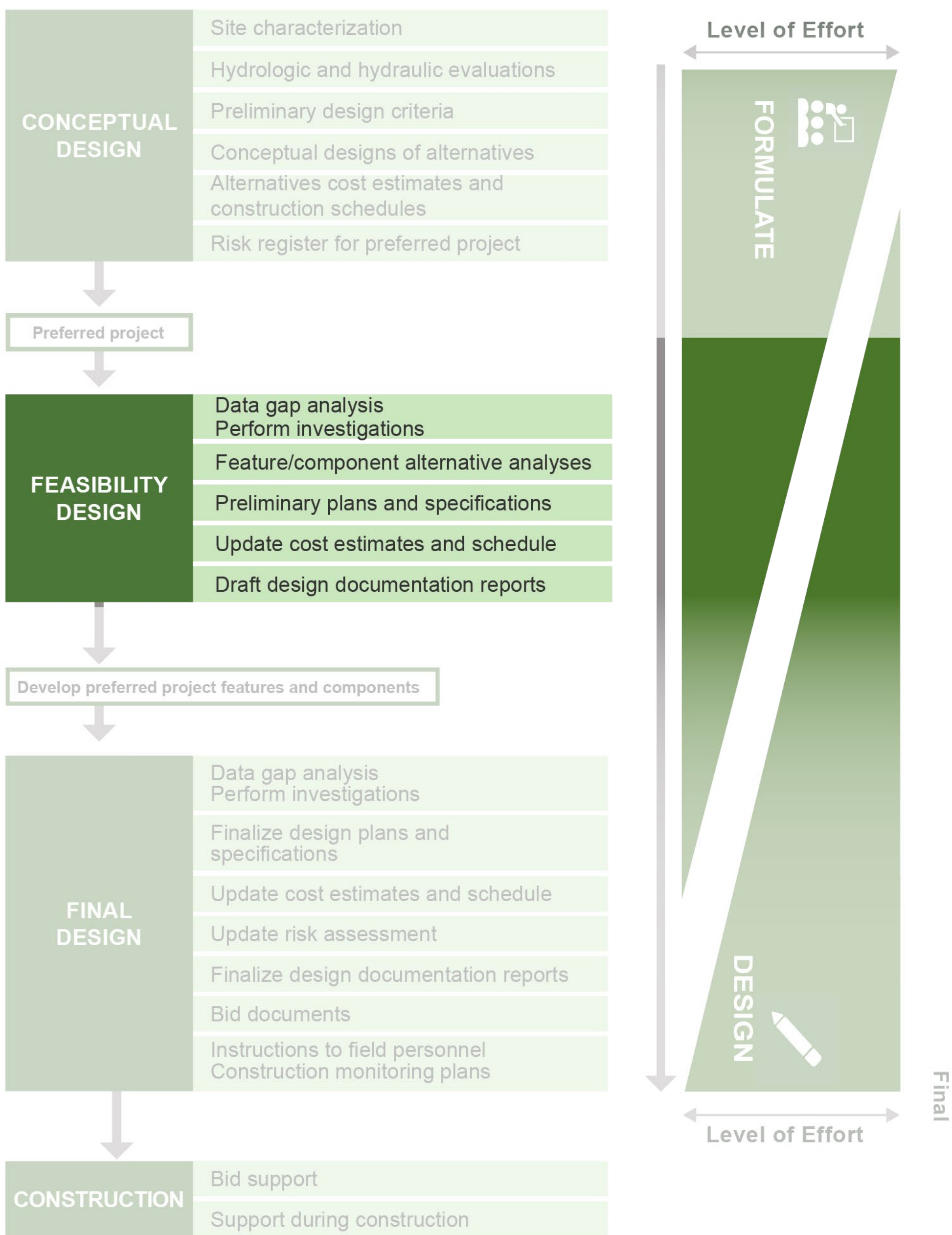
### 2.1.2 Feasibility Design Phase

The conceptual design of the preferred project is advanced during the feasibility design phase (Figure 7-5).

While the levee crest level and alignment are determined as part of the formulation process, the final crest level and alignment may be adjusted during the feasibility design phase (or even final design phase) to meet the requirements of providing the most viable compromise between economy and minimal environmental and social impacts. In this respect, the iterative process of site characterization (section 3) should identify subsurface conditions that would impede the project such as dense or weak foundation layers.

During the feasibility design phase, alternatives for each feature are compared to determine which best accomplishes the project objectives, considering technical feasibility, cost, risk mitigation, resilience, and other factors. A basis of design report is prepared including a list of any anticipated technical specifications (sometimes even including skeleton specifications for key elements of the levee project).

### Figure 7-5: Process for Feasibility Design




Other key activities during this phase are the following:

- **Perform field investigations and site characterizations**. These activities are planned and completed to fill data gaps required to further the analysis and advance the design. These activities may include aerial mapping, site-specific topography and bathymetry, foundation investigation, and utility surveys. A geotechnical data report is prepared.
- **Undergo further refinement of the project's design criteria**. The feasibility design should be risk-informed, scalable, and incorporate resilience where appropriate.

- **Prepare a utility study**. A utility study is prepared to locate all existing utilities in, under, or adjacent to the levee or proposed levee alignment. Utility penetrations may include water mains, sewer mains, agricultural irrigation systems, gas lines, petrochemical lines, and the like. Other utilities may include conduit and duct bank penetrations for electrical and communication lines. The output of the study is a utility and encroachment inventory with planned actions and responsibilities:
  - Utility relocations: It may be necessary to work with utility owners to relocate utilities. It is preferred that utilities run up and over levees and avoid penetrations through levees. Relocation of utilities should start early in the design phase in order to complete relocation work prior to construction, thus avoiding costly delays to the construction of the levee itself. Costs to relocate utilities may have to be included in the project cost estimate. Utilities within or under the levee may require relocation, as well as utilities adjacent to the levee, to provide required clearance for service roads adjacent to the levee toe or for operation and maintenance (O&M).
  - Utility replacements: If relocation is not possible, utilities may require replacement where an analysis and projection of deterioration rates suggest their residual life is less than that of the planned life of the levee, and failure of the utility would present a risk to the levee.
- **Perform ecological assessments**. These assessments of hazardous, toxic, and radioactive waste and draft feasibility design plans may be used to help focus site environmental studies and evaluations in support of federal and state permitting documentation. In turn, the environmental studies may suggest preliminary environmental mitigation measures for incorporation in the design.
- **Update construction cost estimates**. These estimates should be updated at the end of the feasibility design phase, supported by a preliminary construction schedule and a preliminary constructability review.
- **Update the risk register for the preferred project**. The risk register from the formulation study should be reviewed and updated, reflecting any design-related risk reduction measures incorporated into the design, as well as the results of the constructability review.

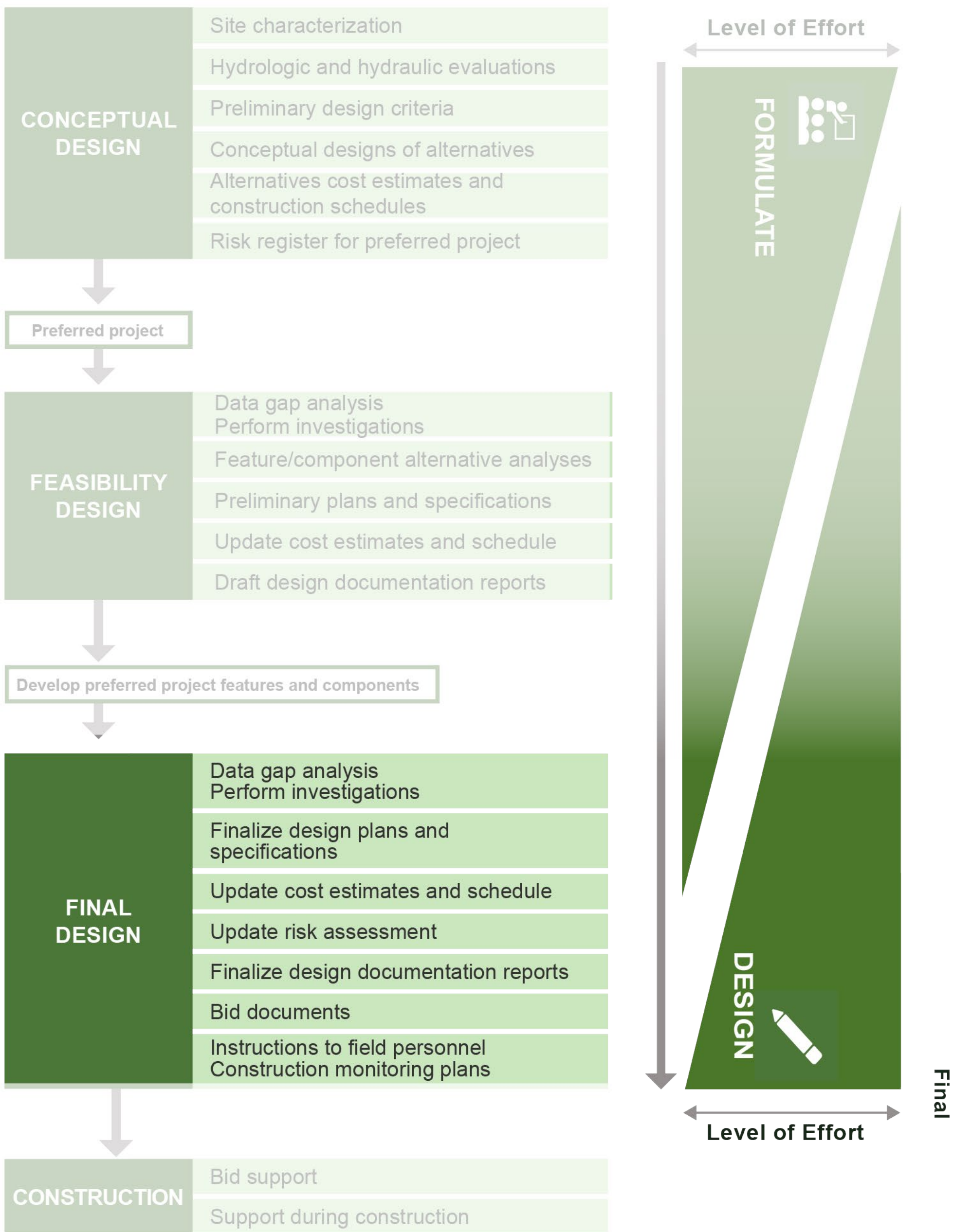
The completion of the feasibility design phase results in a more defined project with project features and components accompanied with draft design documentation reports and updated construction cost estimates and schedules. The cost estimates and schedules are often used for budgeting/funding purposes prior to moving into final design.

### 2.1.3 Final Design Phase

The final design phase (Figure 7-6) takes the selected project configuration from the feasibility design and develops it further to a bid package, ready-to-advertise for constructor quotes, discussed further in **Chapter 8**.

This phase should include final investigations, analyses, plans and specifications preparation, and preparation of support documents, such as a basis of design report or similar. The phase also should include support of the bid process, such as preparing responses to bidder inquiries, preparing bid addenda documents, and assisting the levee owner with bid evaluations.

### Figure 7-6: Process for Final Design




During the final design process, it is best practice to incorporate phased reviews at various levels of design completion. Higher risk levees may need more reviews at different phases for the responsible agencies, levee owners, and stakeholders to review and comment on the progress and quality of the design, as well as verify project goals and objectives are being met. They also provide an opportunity to review project cost and schedule updates and project affordability. Depending on the size of the project and scalability considerations, submittal milestones may include:

- **The 30% design**: Layout plans and details sufficient to define the required features and facilities and their locations, as well as a list of technical specifications and first draft of a geotechnical data report.
- **The 60% design**: More advanced design, including addressing comments on the 30% submittal, draft technical specifications, draft cost estimate, and updated draft geotechnical data report.
- **Draft 100% design**: A substantially complete design with specifications, cost estimate, list of bid items, bid item descriptions, construction schedule for final reviews, and final draft of the geotechnical data report.
- **Ready-to-advertise design package**: Refinement of all project features and assessment of opportunities to optimize design, as well as preparation of construction documents and opinion of probable construction cost.

### 2.1.4 Design Products

Table 7-1 lists the final design products that generally become part of the contract documents for bidding and those intended to provide an information bridge between designers and the construction personnel administering the contract.

**Table 7-1: Final Design Products**

| Products for Bid Support | Products to Inform Construction Personnel |
|---|---|
| Final project plans and specifications (sections 2.1.4.1 and 2.1.4.2) | Basis of design report (section 2.1.4.4) |
| Cost estimate and construction schedule (sections 2.3.2 and 2.3.3) | Designers' instructions for field personnel (section2.1.5.1) |
| Geotechnical data report (section 2.1.4.3) | Construction instrumentation and monitoring plan (section 13) |

#### 2.1.4.1 Project Plans

Project plans, prepared by the designers, are important because they define all of the work to be constructed for a new levee project, or for the modification or rehabilitation of an existing levee. For existing levees, project plans should also include provisions for temporary flood protection measures during construction. The plans should ideally be prepared using computer-aided drafting software. Plan sets are usually submitted to permitting agencies as electronic files or hard copy printed sets. A drafting standard should be established for the project. An example reference for computer-aided drafting standards would be architectural, engineering, and construction computer-aided design standard (USACE, 2019). The plans normally include aspects such as:

- Location information.
  - Project site and feature location plans, and survey control drawings.

  - Real estate information, demolition plans, utility locations, and relocation plans.
- Site characteristics.
  - Geotechnical profiles and cross sections with boring information superimposed.
  - Historic tide gage information and design water surface profiles.
- Details of the levee construction itself.
  - Alignment plans and profiles for the new levee or existing levee to be modified or rehabilitated showing crest and catch profiles, side slope points, berms, working limits, environmental constraints, real estate, and other information.
  - Cross sections, elevations, and detail sheets.
  - Excavation and backfill plans, sections, and details for each levee feature.
  - Borrow areas and haul routes.
  - Features with the levee: mechanical and electrical components, floodwalls, closures, and drainage systems.
- Temporary flood protection measures.
- Other information needed to fully define all requirements for the features and components to be constructed.

Some details may be left to the constructor to design, finalize, or obtain from a supplier, such as details of shoring, staging, and dewatering arrangements, since the constructor is best suited to develop constructability plans. However, initial consideration should be given to such issues during the design process to confirm the basic feasibility of the proposed construction and to facilitate review of constructor submittals (**Chapter 8**). Requirements for submittals should be identified and defined in the documents for construction. Submittals by the constructor should be reviewed and accepted by the designers before the construction work is executed. Selected submittals may also require review by the funding source or permitting agency.

#### 2.1.4.2 Project Specifications

The project specifications should define the technical requirements and include both general and technical specifications:

- General specifications include contractual requirements governing administration of the construction contract and the working relationship between the levee owner, designer, and the constructor.
- Technical specifications for the different project elements should be prepared by the designer. These form the basis for how the constructor will bid and perform the construction work.[1]

The specifications should also set out quality assurance and quality control requirements during construction (**Chapter 8**), as well as required submittals and field approvals.

[1] United Facilities Guide Specifications may provide a helpful starting point for developing these specifications. More information can be found here: https://www.wbdg.org/ffc/dod/unified-facilities-guide-specifications-ufgs.

#### 2.1.4.3 Geotechnical Data Report

The geotechnical data report documents the available subsurface and laboratory data information, including the data obtained under any new investigation. The report normally focuses on information on the index and design properties of subsurface soils. The geotechnical data report describes the following:

- The project and site description based on available site-specific documents and information from prior investigations.
- Additional geotechnical investigation performed, including methods and procedures, location, and depths of borings, in situ tests, the types and frequency of samples obtained, and laboratory test assignments.
- Presentation of compiled data including field boring logs, in situ, and laboratory test results from prior and performed investigations.

The geotechnical data report is sometimes included in the contract documents as a baseline for defining existing conditions.

#### 2.1.4.4 Basis of Design Report

The basis of design report should be compiled to document the design process, analyses, and reasons for key design decisions. It typically includes all design criteria and provides all key design calculations in appendices. This will be an important document during construction, used to verify the design intent and support the evaluation of impacts due to changed conditions. Since risk assessment results (**Chapter 4**) are central to final design, this information and associated decisions about design adjustments, as discussed in section 2.2.1, should also be documented in the basis of design report. This report also will be critical to understanding the project after construction. The report should be provided regardless of the project size.

The basis of design report may include geotechnical aspects, or they can be included in a separate geotechnical basis of design report, which would establish the geotechnical bases of design for the new levee construction or modification/rehabilitation design. The geotechnical aspects included in either report generally include:

- Summary of geotechnical and geologic conditions.
- Characterization of the subsurface materials to establish parameters for design analyses.
- Evaluation of design parameters based on the characterization and analyses results.
- The results of design analyses.
- Remedial design and construction recommendations.

### 2.1.5 Construction Process Support

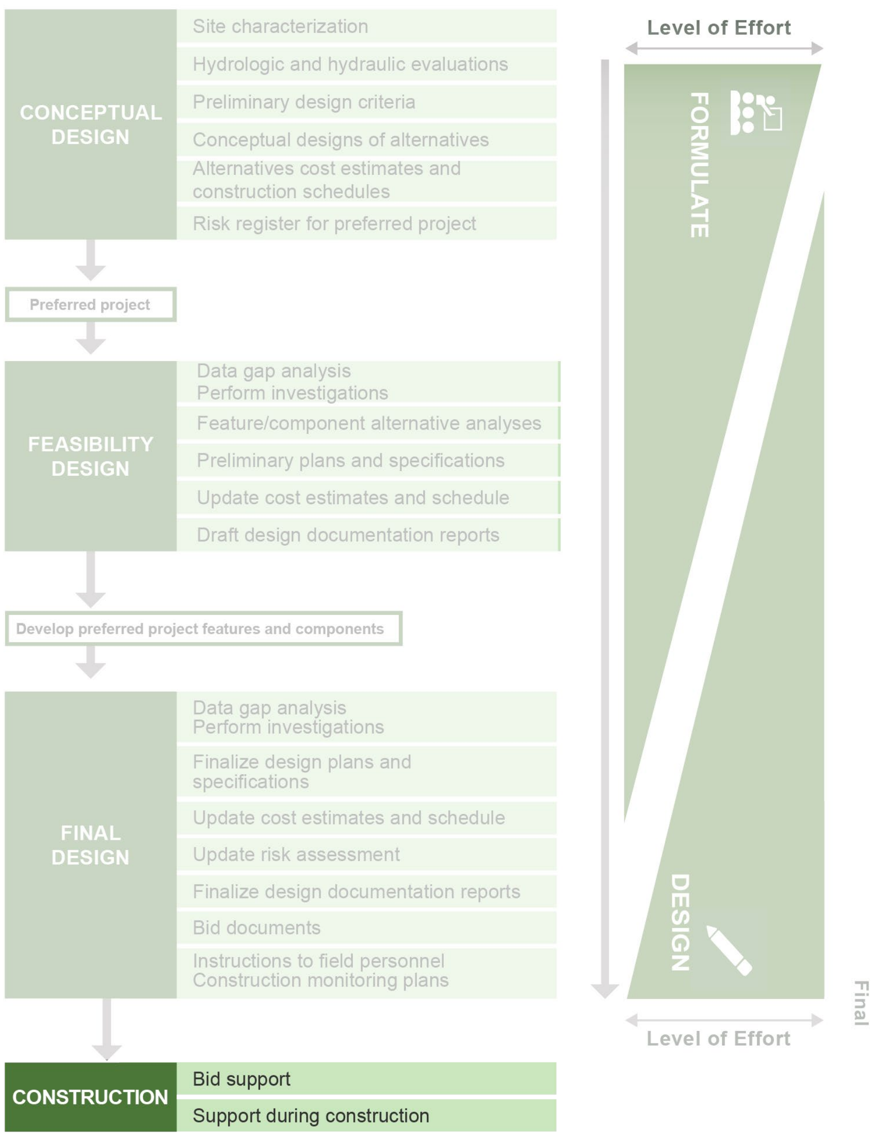
#### 2.1.5.1 General Issues

A successful levee construction project requires a well-defined and clearly understood construction project scope (Figure 7-7).

Defining and conveying the project scope begins during levee design and carries into levee construction, since levee designs should be constructable. Input from levee construction professionals during the levee design process can help ensure the design is constructable (**Chapter 8**). Issues to be considered (including in the cost estimate, described in section 2.3.2) include:

- Assessment of the temporary stability of parts of the levee requiring significant temporary structures or features such as large bracing or dewatering systems.
  - Ultimately, it will be the constructor's responsibility to design any temporary structures or dewatering systems; however, the designer is obliged to consider these issues in order to make sure that the levee is buildable.
- Access for construction machinery or equipment that may require larger dimensions than those necessary for the design of the completed levee.

### Figure 7-7: Support Process for Construction



The levee constructor should understand the important aspects of project design that may require special attention or action during construction. The designer should provide such information and instruction—commonly known as 'engineering instructions [or considerations] for field personnel'—to the levee constructor and the field personnel performing inspections and accepting the work. These instructions should not replace the need for periodic inspections by the designer. Small projects may not require such instructions. The instructions should highlight:

- Design assumptions that should be confirmed during construction.
- Key design elements requiring special attention.
- Required field approvals.
- Other pertinent information for the construction team.

#### 2.1.5.2 Temporary Flood Protection

Ideally, levee work should not be scheduled during known flood prone seasons, although changes in climate mean that such seasons are no longer so well defined. Furthermore, there may be levee **reaches** where temporary flood protection is not practical, in which case specifications should limit construction to outside of the flood season. However, if work has to be scheduled during seasons when flood impacts are a possibility, flooding should be minimized and limited to the extent possible by temporary flood protection measures. The alignment and height of any necessary temporary flood protection should be evaluated as part of the project formulation process (**Chapter 6**).

To mitigate flood risk to leveed areas during construction, temporary flood protection measures should be identified as required, particularly when designing levee modifications or a rehabilitation. Figure 7-8 shows some examples of temporary flood protection for levee construction. These measures should be capable of rapid implementation during construction if the functionality of a section of any existing levee is diminished. The most common potential diminished functionality conditions are:

- A degraded levee crest associated with improvements, or a rehabilitation being made.
- Stripped levee slopes with no erosion or wave action protection.
- Decreased levee section or significant landside excavations.

**Figure 7-8: Examples of Temporary Flood Protection**



*Soil filled bags being used at a levee toe to provide additional flood protection to construction project.*

In designing temporary flood measures, the following issues should be taken into account:

- Seasonality of flooding and the potential changes in flood timing and severity due to climate change (**Chapter 1**).
- Constructability of temporary flood measures during a flood when conditions are bad.

It is also important to bear in mind that levees with diminished functionality downstream from a major dam may be affected by fluvial flows if major releases from the dam are required for operational or emergency reasons.

The selected height and geometry of the temporary flood protection should take account of the likely severe water level/wave events associated with the period of construction. For any given water level, the encounter probability will be lower for the relatively short construction period, and the selected water levels may be lower than those used for the design of the permanent levee system; in this case, a plan for any necessary emergency raising should also be developed. Temporary flood protection planning is discussed in **Chapter 8**.

The temporary flood protection should also be designed such that levee risk (**Chapter 4**) is not increased during construction.

## 2.2 Principles

### 2.2.1 Design Criteria

Design criteria are the explicit goals that a levee project must achieve during the design process in order to be successful (i.e., for the levee project to achieve its intended flood risk reduction benefits). These criteria play a crucial role in shaping the outcome of the levee design process. A general requirement for any levee design is to ensure the levee provides the intended flood risk reduction, including the features and transitions between them as a complete system. This generally requires the levees to be designed to ensure the levee does not breach before it overtops. Resilience of the levee during an overtopping event is also an important consideration during design. A levee design should also be economically feasible and constructable.

For levees, design criteria are synonymous with deterministic standards (e.g., height, factors of safety, limiting values of seepage gradient, minimum dimensions of levee components, etc.) that should be met in order to achieve a reliable and resilient levee. Recommended design criteria for various levee features and components are provided later in this chapter. It is important to note that deterministic standards have the following limitations:

- Deterministic standards are developed from empirical observations for a limited range of conditions that may not be consistent with the local levee project conditions.
- Deterministic standards do not account for every failure mode that can occur on a levee and can overlook critical failure modes.
- Deterministic standards do not explicitly account for uncertainty in the design parameters and methods leading to uncertain levee performance.
- They do not account for planned flood fighting which can significantly affect the levee performance.

### 2.2.2 Risk-Informed and Scalable

The design of new levees, modifications, or rehabilitation of existing levees should use a risk-informed approach that uses a risk assessment to evaluate and adjust the design. A risk assessment can help fill gaps in limitations of deterministic standards to supplement the design process. Implementation of a risk-informed approach therefore involves a two-step process—an initial design followed by a risk-informed design adjustment using a risk assessment.

1. **Perform initial deterministic design**. The initial deterministic design follows the usual design criteria and guidance documents (many of which are cited in this chapter). However, the effort/rigor put into the investigations and analyses should be scaled according to the initial estimates of flood and levee risk (**Chapter 4**). Thus, levees with high potential consequences in their leveed areas will be designed with greater confidence and reliability by reducing uncertainty through more comprehensive investigation and analyses.
2. **Evaluate and adjust design as necessary using a risk assessment**. Since the deterministic design does not tell the whole story, the initial deterministic design should be evaluated and adjusted as necessary according to the assessed risk (**Chapter 4**) associated with that initial design. In higher risk situations, such adjustments may include the addition of complementary resilience measures to increase robustness, redundancy, and recoverability. These may include adjustments that:
   - Ensure the levee will perform adequately for a full range of loadings to the extent possible, ensuring that the levee will not breach before it is overtopped.
   - Ensure risk-driving potential failure modes that remain from the initial deterministic design are adequately addressed.
   - Incorporate additional features that make the levee more resilient without signficant increases in cost.

In lower risk situations, by contrast, a value engineering approach may be adopted in order to remove costly features that are not critical to the performance of the levee. Adjustments may not be necessary for some levee designs.

When adjustments are made to the initial deterministic design, the adjusted design should be reevaluated according to the assessed risk associated with the adjusted design. Further adjustments may be necessary after this reevaluation. In some situations, the initial deterministic design may result in costly design features that are not critical to the levee's performance and, where costs are significant, an adjustment to optimize cost may be necessary. All adjustments including those to optimize cost should be reevaluated according to the assessed risk associated with the adjusted design to ensure the levee meets its design goals and its intended flood risk reduction benefits.

Using a risk assessment to evaluate and adjust a levee design is different than managing implementation of the project—also called project risk—during formulation, design, and construction.

### 2.2.3 Resilience

The concept of community **resilience** is discussed in **Chapter 12**. Designing a resilient levee system involves consideration of robustness, redundancy, and recoverability, taking into account the various areas of uncertainty associated with the levee including:

- The timing and extent of climate-related changes to the hazards over the life of the levee.
- The performance of the levee itself as it changes with time. This includes understanding performance, not just at the selected design condition, but also at lesser and greater conditions, including when the levee is overtopped. This range of performance is typically expressed in the form of fragility curves (**Chapter 4**), which describe the variation in probability of damage/failure as the hazard loading on the levee increases.
- Changes in land use within the leveed area.

With regard to robustness, design criteria should be developed based on the established project flood risk reduction objectives and the increasing uncertainty inherent in the modeling the performance of the levee towards the end of its life. Levees should be designed to accommodate all potential loading conditions, not just the nominal design water level. In particular, consideration should be given to how the levee will perform in cases where the design water level (or wave overtopping rate in the case of coastal levees) is exceeded. In regard to these conditions that are higher than design loading, consideration should be given to measures such as adding levee surface reinforcements at points of first overtopping of fluvial levees. Such reinforcement should be aimed at ensuring that the overtopping which would take place there does not cause erosion and breach the levee. The points of first or preferred overtopping will be determined by considerations of levee superiority (**Chapter 6**).

With regard to redundancy, consideration should be given to adding additional features to enhance the ability of the system to withstand extreme conditions, should one feature fail. Adding natural and nature-based features is one option to increase the redundancy of a levee, as discussed in **Chapter 6** and Bridges *et al.* 2021. Other measures that the community might put in place to limit flood risk are discussed in **Chapter 12**.

With regard to recoverability, consideration should be given to the approaches to be adopted:

- To promptly restore the levee to a serviceable condition in the event of damage.
- To promptly remove excess flood water from the leveed area.

### 2.2.4 Quality Control

Quality control in all phases of design is an important risk reduction measure and is a companion process to the necessary quality control during construction (see Figure 7-9 and **Chapter 8**).

An appropriately staffed and scaled quality control process can help identify and correct errors during the design process. This process should ensure that studies, reports, criteria, plans, specifications, and other technical work products undergo comprehensive and rigorous checking and quality control reviews. A project should have a quality plan that:

- Includes checking and quality control procedures and documentation.
- Takes account of the selection and use of software for design analyses, including the testing and verification of the software itself (either by the designer or a third party).
- Includes verification of important analyses ( as practical) by using more than one method, or more than one computer program, with independent processing of the information and data.

The number and extent of design reviews should be influenced by the results of the risk assessment (**Chapter 4**). When the risk assessment results indicate high risk, independent reviews at interim phases of project design may be implemented as an additional risk reduction measure. Independent design reviews may consist of a consulting board review and/or a constructability review.

### Figure 7-9: Example of Quality Control Testing During Construction



*Sand cone density and nuclear gage testing was performed on the subgrade of the San Joaquin River. An existing drainage ditch adjacent to the landside levee toe was backfilled and a new drainage pipe was installed; April 2021.*

## 2.2.4.1 Independent Expert Review

An independent expert review provides a credible, objective assessment of the levee design. This is important for levee designs where breach or failure of the designed and constructed levee project could lead to loss of life. These reviews often focus on:

- Is the levee design appropriate?
- Did the levee design overlook any critical items?

Independent expert review often occurs during the final design but may begin during the formulation or feasibility phase.

Independent experts should be senior practitioners from outside the design team. They also should be available to assist the design team with other matters, including guidance on site investigations, performing design studies, resolving design issues, working with agencies, and other concerns that may arise. The makeup of the independent expert reviewers should be commensurate with the design features and may include civil, structural, geotechnical, and hydraulic engineering disciplines. Other disciplines such as botanists, biologists, and ecologists should be included for natural and nature-based features. For smaller projects, one reviewer may be appropriate.

The reviewers typically continue reviews into the construction phase, providing continuity between design and construction. In addition to periodic site visits during construction, the reviewers can also provide the levee owner, design team, and construction management personnel with suggested guidance on managing construction issues, such as changed conditions or problems meeting specification requirements.

#### 2.2.4.2 Constructability Reviews

Constructability reviews are an important part of the risk reduction strategy for a project and should be included in the schedule for the final design. In this review, bid documents are to be reviewed by a qualified construction specialist or team if appropriate, based on the features and components of the project. The goals should be to verify completeness, constructability, coordination of documents, and a clear presentation of requirements for bidding and execution of the work. The review can be applied to new levee projects, as well as to levee modification and rehabilitation projects.

The qualified construction specialist should be a senior practitioner who is familiar with the elements to be constructed, as well as with applicable construction techniques and practices. The specialist may be associated with the engineering firm performing the review who is not involved with the design or associated with a separate firm under contract with the levee owner for providing comprehensive construction management services. Some of the benefits of a comprehensive constructability review include:

- Potential for receipt of more bids.
- Receipt of more confident, lower bid pricing.
- Fostering good working relationships between the levee owner, designer, and constructor.
- Reduced chances for changes, claims, and disputes during construction.
- Better prospects for delivery of a quality finished product, on schedule and within budget.

## 2.3 Considerations

### 2.3.1 Selecting a Levee Designer

Selecting the appropriate levee designer is an important decision to ensure successful completion of a levee project. The levee designer should be experienced in design of levees and familiar with local conditions and requirements. For complex levee projects, the designer is often made up of a team with appropriate technical disciplines to characterize the site and design the levee features. Typical levee designer functions include:

- **Project management**: Managing and controlling all aspects of the project, including budget, schedule, and quality.
- **Technical leadership**: Technical analyses and design of the levee features.
- **Technical support**: Generally including hydraulic, geotechnical, structural, and civil engineering, along with cost estimation. Other disciplines may include, but not be limited to, surveying, environmental science, geomorphology/geology, data management, computer-aided-design, cultural resources, landscape architecture, and mechanical, electrical, and hazardous materials engineering.

### 2.3.2 Cost Estimating

In the early formulation and design phases, cost estimating provides essential input to the decision-making process, particularly when evaluating alternative components. In the later design phases, cost estimating should become more comprehensive, supporting financial planning for the project and providing a baseline to track and control construction costs. Cost estimating should also cover O&M costs.

Cost estimates for levee projects should be:

- Comprehensive, well documented, accurate, and credible.
- Developed to a degree of confidence and accuracy appropriate to the level of completion of the levee design. As a corollary, the quality, reliability, and level of completion of the design that forms the basis of the costing should be commensurate with the expected accuracy of the cost estimate.
- Performed or updated within a reasonable time of their intended use.

#### 2.3.2.1 Estimate Components

A complete project cost estimate typically includes construction contract costs and non-construction contract costs (including O&M costs), both of which need to be considered in financial planning for the project. Thus, allowances for construction contingencies (e.g., changed site conditions, change orders, and claims) should be added to the anticipated bid price at the time of appointing the constructor. Refer to  for additional guidance regarding contingencies.

The non-construction contract costs are real costs over and above the constructor's contract cost that need to be part of the overall financial plan for the project. Some of these costs include:

- Licensing and permitting costs.
- Environmental mitigation costs.
- Site characterization costs, including surveying/mapping, geologic and geotechnical investigations, laboratory testing, and data analysis.
- Engineering and design, including plans and specifications, supporting design, and geotechnical reports.
- Construction management services, including the construction manager, inspectors, and testing laboratories.
- Levee owner administrative and staff costs during design and construction.
- Temporary and permanent property acquisition costs and public utility relocation costs.
- Long-term O&M costs.

#### 2.3.2.2 Estimating Methods

In the conceptual and feasibility phases of a project, judgment and parametric modeling may be used because of the lack of design detail. Parametric (or stochastic) modeling uses available cost information from other similar projects or for similar types of work and scales these costs up or down to reflect differences between the similar project and the new project. For example, if a cost per mile is known for an existing embankment levee or floodwall, it can be applied to the planned work after adjusting for regional cost differences and any differences in height and width. This sometimes is referred to as a top-down estimate. The judgment of a qualified cost estimator should be taken into account in using the existing cost data.

In the later phases of design, detailed quantity estimates should be made for each work item (e.g., place and compact fill, form and place concrete). The unit costs then should be developed by considering material, labor, and equipment needed to complete work items. This is referred to as a deterministic or bottom-up estimate.

> **SOURCES FOR COST ESTIMATE CLASSES**
>
> - American Society for Testing and Materials International E2516, Standard Classification for Cost Estimate Classification System (E06 Committee, 2019).
> - Association for the Advancement of Cost Engineering International, Recommended Practices, 17R–97 (AACE International, 2020), 18R-97 (AACE International, 2020), and 56R-08 (AACE International, 2020).
> - U.S. Army Corps of Engineers, Engineering Regulation 1110-2-1302, Civil Works Cost Estimating (USACE, 2016c).

#### 2.3.2.3 Estimating Software

Cost estimating software packages reduce the time required to prepare cost estimates and can help to increase the accuracy of estimates by enabling the utilization of proven standard formats, processes, and procedures. If possible, the same estimating package should be used through various phases of the project to facilitate efficient transfer of cost estimate information. In some cases, the funding agency often specifies requirements for cost estimating software. In general, cost estimating construction cost software should contain:

- Standard formats, processes, and procedures.
- Ability to easily update labor, material, and equipment costs (unit prices).
- Flexible report writing.
- Area cost factors for specific location of the construction activity.
- Historical construction cost data.
- Cost risk analysis.

#### 2.3.2.4 Cost Estimate Accuracy and Class

Construction cost estimate accuracy depends primarily on the maturity of design detail available when the estimate is prepared. As design details are refined, the cost estimate becomes a more detailed bottom-up estimate, with less reliance on contingencies. Figure 7-10 shows the improvement in accuracy of a cost estimate through the various phases of design (from conceptual to final design) according to a class based on design maturity.

Table 7-2 (adapted from the sources in the callout box) shows a typical, commonly used estimate classification system for process and general building construction industries, based on percent of design completion. The table also shows the intended use for the estimate in each class, methods used to develop the estimate, and the typical contingencies for each class.

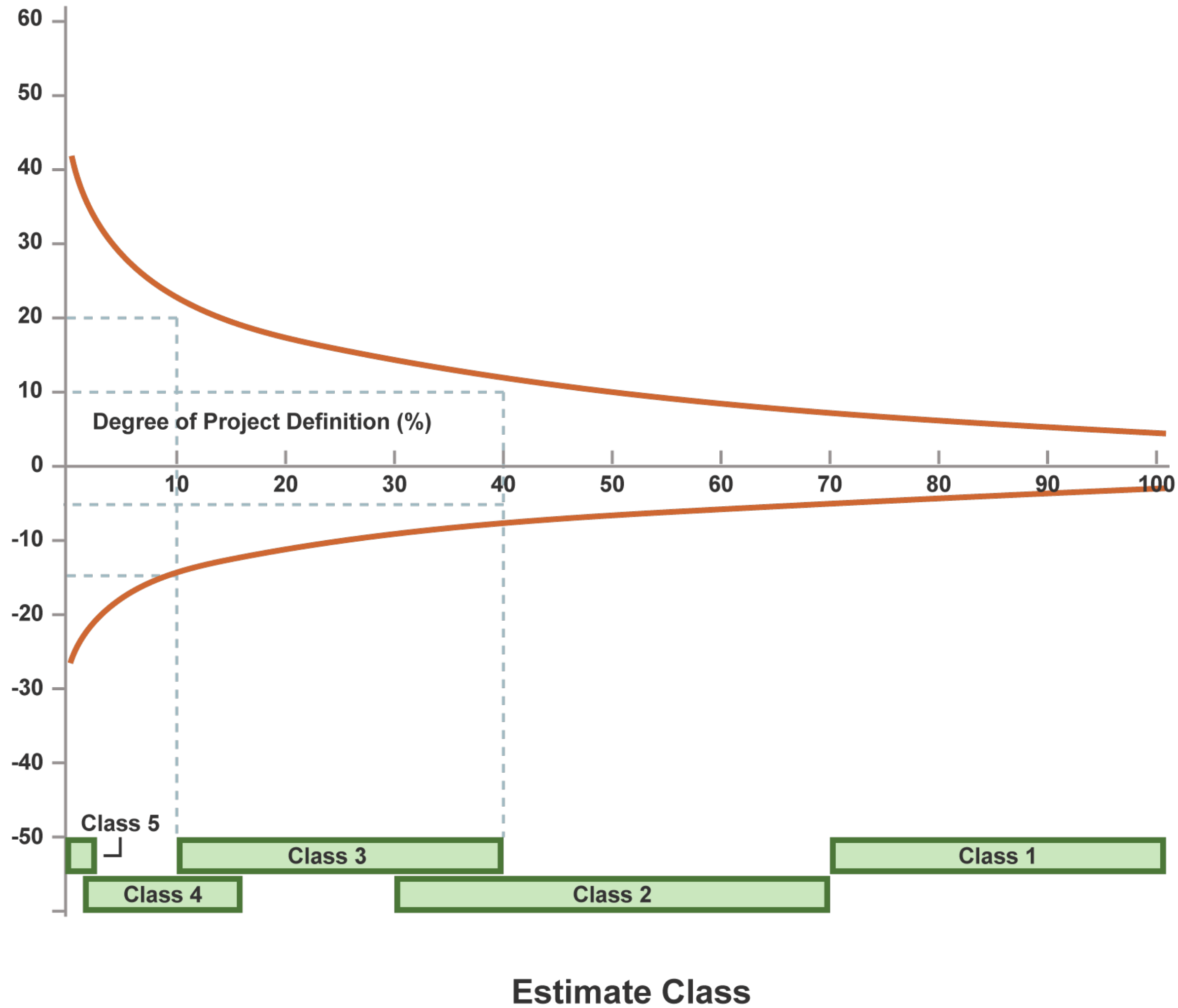
### Table 7-2: Typical Accuracy Ranges for Construction Cost Estimates

| Estimate Class | Maturity of Design[1] (percent) | End Use | Estimating Method | Typical Contingency[2] |
|---|---|---|---|---|
| Class 5 | 0% to 5% | Concept screening | Judgment, parametric model | 40% to 200% |
| Class 4 | 5% to 10% | Study or feasibility | Factored costs, parametric model | 30% to 100% |
| Class 3 | 10% to 60% | Budget authorization or control | Semi-detailed unit costs, assembly level take-offs | 20% to 50% |
| Class 2 | 60% to 80% | Control or bid/tender | Detailed unit costs, detailed take-offs | 15% to 30% |
| Class 1 | 80% to 100% | Check estimate or bid/tender | Detailed unit costs, detailed take-offs | 5% to 15% |

Notes to table:

1 Maturity of design as a percentage of a complete 100% final design.
2 Range of contingencies to achieve 80% confidence level in the cost estimate (common industry standard). A schedule cost risk analysis, as outlined in the Cost Guidance and Schedule Risk Analysis Guidance (USACE, 2009), can be used to inform the level of contingency for the project.

**Figure 7-10: Estimate Accuracy Improves with Time**



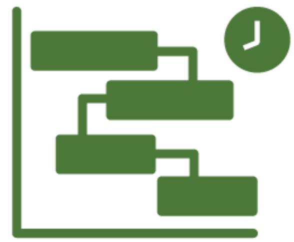

As design maturity increases, the end use of the estimate progresses from strategic evaluation and feasibility studies to funding authorization and budgeting, and then to project budget control.

- Class 5 estimates are based on preliminary technical information and are often referred to as rough order of magnitude estimates.
- Conceptual phase estimates would generally be Class 4 or Class 3, based on design maturity and the requirements of the funding agency.
- Feasibility study estimates are typically Class 4.
- Final design estimates can be Class 1 or Class 2, as required by the funding agency.

## 2.3.3 Easements and Permits

Easement and permit requirements identified during the project formulation phase (**Chapter 6**) should be confirmed during design.

### 2.3.4 Project Implementation Scheduling



The purpose of a project implementation schedule is to:

- Define distinct tasks that should be executed in proper sequence to successfully complete the project.
- Prepare an accurate cost estimate (section 2.3.2.2); thus, the schedule becomes a cost risk management tool.

A project implementation schedule typically covers the project from conceptual through final design, permitting activities, construction, and project closeout. The initial schedule usually is at a high level and becomes more refined throughout the project life, as project details are finalized.

Scheduling usually commences by breaking tasks into manageable work elements, referred to as the work breakdown structure. Each task is assigned a duration—usually based on past experience, known duration rate, or other reasonable assumptions. Links between tasks are added that control when tasks can begin or should end to achieve milestone completion dates. Tasks can be resource/cost loaded (e.g., manpower, equipment) to assist in resource allocations, budget planning, and budget control. Some benefits from scheduling include:

- Providing schedule duration input for preparing cost estimates and evaluating escalation.
- Supporting budget development and budget control for tasks and for the entire project.
- Guiding resource management and the allocation of resources to complete tasks.
- Providing a tool to track progress and identify schedule delay issues.
- Allowing stakeholders to follow project progress.

Commercially available software packages are available to prepare schedules. They should be evaluated to determine which are best suited for the project. Some permitting and funding agencies specify the software to be used. Because the schedule will be a living document that will need to be updated periodically throughout the formulation and design process, schedule software should not change.

### 2.3.5 Vegetation

Having the appropriate vegetation type and approach for long-term management on and near the levee is essential to ensuring the levee performs, operates, and is maintained as intended. For example, the type of vegetation at the levee overtopping location may have a positive or negative impact on levee performance. Herbaceous grasses or forbs that have dense root systems may provide some erosion resistance that reduces the potential for breach during some overtopping events, while the presence of a lone tree or the absence of consistent vegetation may accelerate erosion during an overtopping event.

In general, desirable characteristics of vegetation management on or near levees include:

- Does not inhibit access for visual inspections, especially during flood events.
- Avoids activities that can damage levee features and lead to poor levee performance.

- Ensures appropriate ground cover is used to reduce soil erosion.

Characteristics of desirable species include:

- Requires little or no mowing.
- Able to resprout (i.e., perennials).
- Have fine, deep fibrous roots which will lend strength during an overtopping event and hold the vegetation and soil in place.
- Are unpalatable to local burrowing animals.
- Will withstand the seasonal climate and weather, including drought tolerance if relevant to the local environment.
- Are salt tolerant as appropriate to the expected levels of salinity (for levees in coastal areas).
- Are fire resistant (unless controlled burns are part of the maintenance plan).

For most situations, herbaceous vegetation (e.g., grasses, wildflowers, and forbs) satisfies these desirable characteristics. However, other types of vegetation on or near the levee may be needed to satisfy the levee project objectives such as environmental benefits, laws, and regulations. These types of vegetation should only be used on or near a levee when there are intentional design elements to accommodate this vegetation.

When considering the appropriate vegetation type and spacing of plantings during design, experts—such as scientific professionals and tribal experts (if applicable) well versed in sediment transport, fluvial geomorphology, fish biology, botany, forestry, ecology, and soil science, in addition to the traditional engineering team—should be included in the design process. It is vital to ensure all maintenance tasks, including any vegetation maintenance tasks, are evaluated and deliberate (or designed). As such, vegetation planting and/or management are included in the O&M manual to ensure that all maintenance actions can be effectively carried out without undue regulatory hinderance.

Designed vegetation elements (e.g., other than grasses), which allow vegetation to thrive without compromising the levee or creating a maintenance burden, can be incorporated. Designed vegetation elements to incorporate shrubs, trees, or other woody plants in a deliberate manner can enhance the environment while reducing uncertainty of compromising levee performance. These vegetation elements should be designed considering the following:

- Levee features are not damaged or perform poorly due to the vegetation throughout the life of the levee (this includes considering potential impacts of vegetation growth, maturity, and death).
- Long-term maintenance requirements can be reasonably and satisfactorily performed.
- Necessary access, inspection, and floodfighting are not hindered.
- The required confidence and reliability of the levee is achieved.

Examples of vegetation design elements that accommodate types of vegetation other than grasses on or near levees include levee setback, enlarged levee embankment, planting berm or bench, and/or planting boxes.

### 2.3.6 Encroachments and Penetrations

**Encroachments** include any activity on or physical intrusion on, over, through, or under the levee that is not related to the flood risk reduction benefits or other co-benefits the levee is intended to provide. Examples are buildings, fences, pipelines, and other utilities. Where possible such encroachments should be avoided, but where necessary the design should be adjusted to limit the impact on levee performance.

**Penetrations** are a subset of encroachments which, as they pass through or beneath a levee. are of particular significance for levee performance. Design issues to be addressed in such situations include:

- Leakage from pipe penetrations.
- Differential settlement adjacent to the penetration.
- Seepage and internal erosion along the outside of the penetration.
- External erosion due to water flow around the penetration as it passes into the levee.

Further information on pipes is provided in section 11.1 and detailed guidance on designing for levee penetrations is available in Engineer Manual (EM) 1110-2-2902 (USACE, 2020b).

### 2.3.7 Challenges

Despite their apparent simplicity, the design of levees can be surprisingly challenging compared to what would appear to be more complex structures. Table 7-3 gives a summary of the main considerations that help to address the challenges that arise during the design process.

**Table 7-3: Main Challenges of Levee Design**

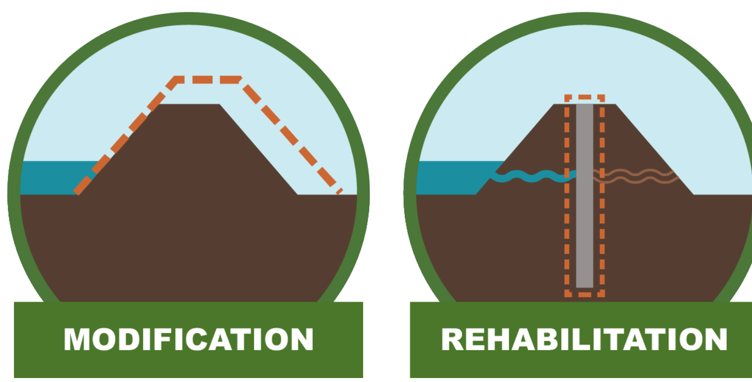
| Design Challenge | Chapter or Section | Summary of Associated Design Considerations |
|---|---|---|
| Site characterization and its impact on design | Section 3 | • Existing information and collecting new information about site conditions is critical. Required information includes topography, bathymetry, geology, geotechnical, hydrologic and hydraulic data, utilities.<br>• Information gathering and interpretation occurs in every phase of formulation and design.<br>• Soft alluvial foundation soils need special attention.<br>• Materials taken onsite are prone to variability, imperfections, and deterioration with time. |
| Design coordination with construction | Section 2.1.5 | • Coordinate on variability in site conditions and construction materials.<br>• Coordinate regarding aspects that require special attention or action during construction, including instrumentation and monitoring. |
| Levee reach selection | Section 3.3.6 | • Established to support analysis and design of the levee.<br>• In highly variable ground conditions, a potentially large number of reaches may be necessary for analysis. |

| Design Challenge | Chapter or Section | Summary of Associated Design Considerations |
|---|---|---|
| Levee crest level | Chapter 6 | • Level should be set to provide the planned flood risk reduction throughout the design life.<br>• Superiority, wave runup, potential settlement, and resilience should be considered in establishing the level. |
| Temporary flood protection | Section 2.1.5.2 | • Should be included in all levee designs, particularly where the project construction schedule will cover multiple years.<br>• Applies to coastal, as well as riverine levee systems, and locations where internal drainage could adversely impact levee construction on the landside. |
| Closure structures | Section 7 | • Design of these structures needs to avoid introducing points of vulnerability into the levee system. |
| Transitions and other points of weakness | Section 8 | • Need to avoid loss of integrity at the transition point between different features, which could lead to failure of the levee at these points. |

## 2.3.8 Levee Feature Modifications or Rehabilitation

Levee **modifications** include activities that change the original operation and function of a levee. It includes raising a levee, modifying its alignment, or changing features. Levee **rehabilitation** includes activities that restore a levee to its original operation and function due to two possible causes:

- Extensive deterioration, including damage by high water or other incidents.
- Deficiencies from design/construction.




Rehabilitation is more substantial than normal maintenance and repair and is not routine in nature. Rehabilitation should restore the features to add serviceability and design life.

The equivalent of conceptual and feasibility phases for design of such modifications or rehabilitation for an existing levee is a phased evaluation process of the existing structure to determine the necessity of the modifications or rehabilitation. While the principles and best practices for design of a new levee are still generally applicable, the evaluation process should include the following steps:

- Evaluation of existing and future conditions (including climate change and development in the floodplain), which may have increased the hazards faced by the levee and the consequences of failure.
- Development of acceptance criteria based on back-analysis of historical performance where existing structures do not meet current codes or guidelines.

- Analysis of the existing levee using methods that accurately describe its behavior without introducing excess conservatism.
- Identification of rehabilitation options.
- Repeat analysis of the levee performance for each option.
- Selection of a preferred option—likely making use of a decision matrix—including life safety, cost, environmental and community impacts, and benefits for formulation, evaluation, and selection.

Note that existing levees may have performance information that can be used to identify potential deficient areas and target modification efforts. The amount and quality of performance data is highly variable across different levees. When available, this information can be used as part of a risk-informed approach, to prioritize levee modification projects and select appropriate design criteria.

The level of analysis for performance evaluation should be commensurate with the decision being made. Analyses should reduce uncertainty enough to allow confident decision making. The analysis process for levee evaluation is typically phased, and the need for each phase is dependent on the outcome of the previous phase.

- **Preliminary analysis**. This is performed based on available data and the actual conditions of the levee. Before performing an analysis, the available existing data and information about the levee should be collected and reviewed, including geologic and foundation data, design plans, as-built plans, periodic inspection reports, damage reports, plans of previous modifications to the levee, and other pertinent information. The designer should inspect the existing levee to assess its condition.
- **Comprehensive analysis**. If the preliminary analysis indicates the existing levee does not meet safety and performance objectives, a plan of action for a comprehensive evaluation should be developed. The plan should determine the extent of the exploration and testing program needed to accurately define soil parameters, the analytical program needed to accurately define the loading conditions, the remedial schemes to be studied, and the extent of any additional parametric studies. An exploration, sampling, testing, and instrumentation program should be developed to determine the magnitude and reasonable range of variation for the parameters that have significant effects on the safety and performance of the levee, as determined by parametric studies. Analysis of the levee should be performed using the material properties and strength information obtained from the sampling and testing program.
- **Advanced analysis**. If the stability of the levee is still in question after completing preliminary and comprehensive analyses, advanced analytical studies should be considered to reduce uncertainty where the risk and/or cost of remediation is high. These studies should use two- and three-dimensional finite element methods to capture the interaction between the foundation, backfill, and the structure, and to capture the capacity of the structural system to distribute loads to adjacent monoliths and abutments.

# 3 Site Characterization

The goals of characterization are to identify hydraulic, geotechnic, and morphologic design requirements and constraints, as well as to establish design parameters. Developing an understanding of the system characteristics to facilitate a design that meets the project objectives will require consideration of the available information and additional data that will improve confidence in the design.

Levee alignments frequently traverse varying conditions and could fail at the weakest location. Accordingly, designs should characterize the full alignment length and identify reaches that may be critical. A balance should be established between investigation expenditures, construction costs, and risk reduction objectives according to the degree of risk associated with the levee (**Chapter 4**). Increased investigation costs will reduce uncertainty, increase confidence, and may reduce construction costs by decreasing design conservatism. However, this reduction will not be linear and full characterization of conditions at every location is not practical.

Existing data should be evaluated to characterize the site. Then, additional investigation to improve design confidence, reduce construction costs, and better understand flood and levee risk can be planned. This should be an iterative process and can be most efficient when performed in phases. Even so, project budget and schedule needs may require eliminating phases.

Characterization activities generally include existing information gathering and review, interpretation, and data gap analysis. These activities are not linear because existing information gathering and review is a one-time process, while interpretation and investigation frequently are performed in phases, sometimes correlated to the design phases (i.e., conceptual, feasibility, final).

## 3.1 Gathering and Reviewing Existing Information

Information gathering and review should be performed to understand available information and to characterize the alignment to the extent possible. This should allow evaluation of opportunities for investigation to improve design confidence. The data gathering and review process should involve collecting and cataloging existing information pertinent to a flood risk reduction project. The data will be useful throughout the project, including during formulation, risk assessment, and other levee lifecycle activities.

### 3.1.1 Information Gathering

For project design, the information will be used to characterize site conditions and develop design parameters. Typical documents to review should include:

- Results of risk assessments.
- Documents for existing levee features.
  - Data from the National Levee Database (NLD) for existing levees.
  - As-built drawings and reports of existing features.
  - O&M manuals.

    - Inspection reports.
    - Performance history.
    - Geophysical test results.
- Existing geotechnical data from other projects designed/constructed in the area.
    - Existing publicly available information.
    - Existing hydrologic and hydraulic data, water level gage data, and tide gage data.
    - Topographic and bathymetric maps.
    - Geologic and geomorphic maps.
    - Information on, and classification of, the groundwater regime.
    - Published papers, reports, and available information from local, state, and federal agencies, such as the United States Geological Survey and the U.S. Army Corps of Engineers (USACE).
- Information from the formulation phase.
    - Environmental, cultural, and real estate studies.
    - Levee siting concerns or constraints received from community members or other stakeholders.

The information gathering and review process should be fluid and ongoing throughout design, as data gaps and analysis needs are identified. The purpose of the process should be to collect and assimilate sufficient data at each design phase to inform required decisions, culminating in the successful final design of a levee meeting project objectives.

### 3.1.2 Data Storage

The framework for data storage should be developed during the formulation phase of the project (**Chapter 6**), for use by the project team in each step of the design process and allow additional information and data to be added as it is collected. This database can be used to evaluate data gaps to be filled during the investigation phase. Geographic information system (GIS) databases, particularly for larger or more complex projects, offer one way of providing systematic storage of collected project information and data. GIS allows for easy access and review, which is essential to facilitate the design processes and for project documentation. Relevant data should also be uploaded to the NLD (**Chapter 4**).

## 3.2 Additional Investigations

The assembled data should be reviewed to assess whether sufficient data is available for design. The design data requirements will vary based on the design phase, with conceptual design requiring the least data and final design the most. Data requirements should be risk-informed and consider uncertainty and conservatism. The degree of conservatism and acceptable levels of uncertainty should be informed by the results of the risk assessments. The investigation process should be undertaken to fill identified data gaps, to move the project design forward. Factors to consider in developing an additional investigation program include previous experience, proposed levee height and side slopes, likely foundation conditions (geology and geomorphology), likely duration of high-water events, and the nature of available borrow materials. For existing levee projects, additional factors include construction history, levee conditions, past performance, topographic/bathymetric anomalies (e.g., depressions in toe blankets), presence/nature of structures and utilities in embankments, and extent of mitigation measures.

The extent and scope of investigations will vary over a project's lifecycle, as data needs increase and project funding becomes available. Additional investigation may cover many different aspects, but the most common will be:

- Topographic survey and bathymetry
- Inspection and testing of existing levee features
- Geologic and geotechnical investigations
- Hydrologic and hydraulic data collection, water level gage data, and tide gage data
- Utility surveys
- Hazardous materials surveys

Figure 7-11 shows typical field investigation of foundation conditions through the collection of boring samples.

**Figure 7-11: Example Field Investigations**

*Drill rig with hollow stem auger used for extraction of soil cores at Dawson Field within the U.S. Department of Agriculture Research Center, Georgia.*

## 3.2.1 Data Gap Analysis

The data gap analysis should review all available data and determine where additional information is needed to support design  and to minimize design and construction risks. As mentioned above, the determination of requirements for additional data should be informed by the analyzed flood and levee risks (**Chapter 4**). The risk assessment results will aid in the identification of acceptable levels of uncertainty for the levee project and the selection of the appropriate degree of conservatism.

The timing of the data gap analysis will be important to allow sufficient time to obtain permissions for property access and necessary permitting activities. Gap analysis should be ongoing, but should be performed specifically on two occasions:

- Near the end of the conceptual phase to support data gathering for the feasibility phase.
- Near the end of the feasibility phase to support data gathering for final design.

### 3.2.2 Topography and Bathymetry

Topographic survey and bathymetry should establish baseline grades, critical for hydraulic analyses and the design of levee features. Drone surveys may be beneficial in the formulation and design phases and should be considered.

Control and topographic survey accuracy and data collection will be important. EM 1110-1-1005 (USACE, 2007) provides guidance on planning and executing a survey program, survey datums, accuracy requirements, and other topics. EM 1110-2-1003 (USACE, 2013) provides useful guidance on performing hydrographic (bathymetric) surveys, including datums, methods, and accuracy. The project datum should be established and consistently used throughout the design and construction process.

Conceptual design may be performed with less topographic coverage, provided sufficient understanding is available to assess required features. As the design progresses, additional survey and bathymetry may be required to refine design features and meet requirements. During geometric interpretation, locations where insufficient topography is available, or the topography lacks sufficient detail should be identified for additional topographic surveys and/or bathymetry surveys.

Also, visible aspects of buried utilities (e.g., pipe inlet/outlet elevations, pull box, and manhole locations) should be located by ground surveys as part of developing topographic mapping for the project.

### 3.2.3 Geotechnical and Geomorphic

The purpose of the geologic and geotechnical investigations is to characterize subsurface conditions that impact levee performance and design. Investigations can be costly and time-consuming; therefore, they should be carefully planned to optimize the information obtained. The purposes of the geologic and geotechnical investigations should include:

- Characterizing foundation conditions along the levee alignment and adjacent areas.
- Characterizing existing levee features, including embankments and berms.
- Obtaining geological and geotechnical data to develop design analyses parameters.
- Characterizing groundwater conditions and their seasonal variability for project features and borrow areas.
- Developing reach and sub-reach boundaries (section 3.3.6).

#### 3.2.3.1 Geotechnical Investigation Planning

The planning of geotechnical investigation should be informed by any prior risk assessment. Areas of higher hazard flood risk and/or levee risk (**Chapter 4**) generally should have a higher intensity of geotechnical exploration, characterization, and analyses. In addition, the focus on analysis of probable failure modes should dictate exploration locations and depths.

A comprehensive geologic and geotechnical investigation plan should be developed, considering potential failure modes, site-specific conditions, cost, permitting, and coordination. A written comprehensive plan should be developed for field investigations, to justify the selection of exploration techniques, locations, sampling plan, and depths.

Issues to be taken into account in developing spacings of borings include:

- Need for borings at the crest location and at the landside and waterside of the levee.
- Verification of cone penetration testing with conventional material recovery borings and standard penetration testing.
- Appropriate spacing of borings along the levee alignment (section 3.2.3.4).

Issues to be taken into account in determining the depth of any boring include:

- Identification of the uppermost low permeability layer.
- Definition of aquifer characteristics.

Carrying out sufficient geotechnical borings is a project risk reduction measure that better informs design. Geotechnical boring methods should be selected based on the expected geologic conditions, required exploration depths, sample requirements, and project budget and scale. The boring plan should accommodate the levee feature being considered (embankment, floodwall, closure structure, or transitions). Table 7-4 presents some key considerations when utilizing geotechnical borings.

### Table 7-4: Geotechnical Borings Considerations

| Item | Investigation Goals | References |
|---|---|---|
| Exploration locations, spacing, and depths | The location, spacing, and depth of boring and cone penetration test explorations should be risk-informed and project-specific and/or from previous experience in the area. Typical spacings along the levee alignment will be between 200 to 1,000 feet, being more closely spaced in expected problem areas (such as areas of poor past performance or locations of critical geologic features like oxbows or recent channels) and more widely spaced in expected less-problematic areas (such as older geologic formations without past performance distress). | EM 1110-1-1804 (USACE, 2001a), EM 1110-2-1913 (USACE, 2000) |
| Sampling and laboratory testing | The purpose of sampling should be to log and characterize levee stratigraphy and obtain samples for laboratory testing, to aid in developing property parameters for analysis and design. | ASTM International standards generally |
| Groundwater measurements | Groundwater levels, if encountered, should be measured during explorations and monitored to provide information needed for design. | EM 1110-2-1908 (USACE, 2020a) |
| In situ testing | In situ tests often are the best means for determining the engineering properties of subsurface materials and, in some cases, may be the only way to obtain meaningful results. | EM 1110-1-1804 (USACE, 2001a), TM 5-818-5 (Departments of the Army, the Navy, and the Air Force, 1983) |

An engineering geologist or geotechnical engineer with levee drilling experience should be assigned to drill rigs, to oversee the work, to classify soils onsite, and prepare field drill logs. Caution is required when drilling in levees to avoid damage (e.g., hydrofracturing). Engineer Regulation (ER) 1110-1-1807 (USACE, 2023) provides a good outline of the drilling program plan. Permits may be required for drilling in existing levees. Time for this approval process should be incorporated into the schedule for the field work.

Geotechnical analysis should consider uncertainty in parameters and stratigraphy. Sampling and laboratory testing is intended to reduce this uncertainty, lowering project risks and costs. The sampling and testing program should be specifically designed to accomplish this purpose.

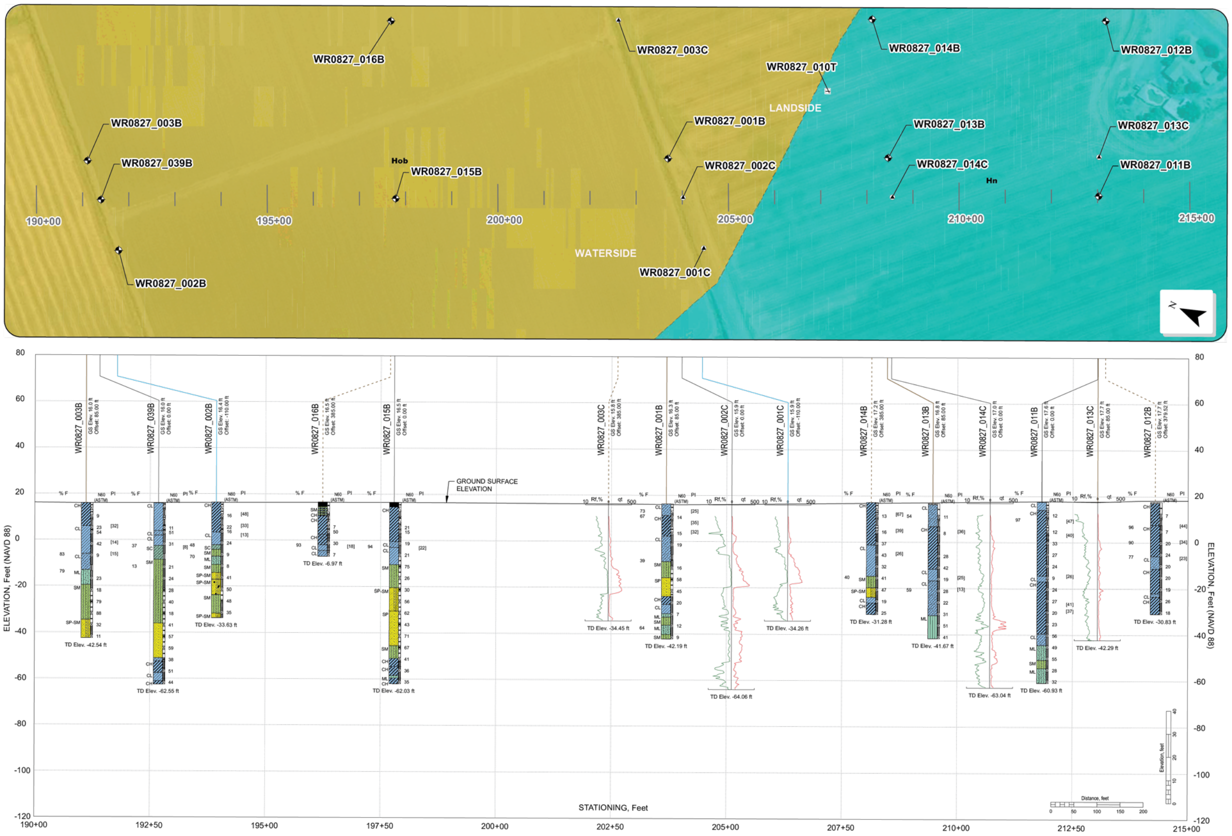
Table 7-5 summarizes goals and the extent of investigations in different phases of formulation and design. The data required for each design phase will vary and be progressively more intense. Geotechnical investigation should be performed in phases for larger and more complex projects. This phasing should allow the review of information that is obtained to inform further investigations, as well as allow more targeted investigation for specific design features as the design progresses.

### Table 7-5: Investigation at Various Project Phases

| Project Phase | Investigation Goals | Intensity of Investigation |
|---|---|---|
| Problem identification | Existing conditions characterization | Low: Widely spaced explorations, may rely on geomorphologic and geologic mapping or historical reports. |
| Formulation | Inform formulation level design; identify constraints | Low: Confirm expected geologic conditions and investigate potentially problematic geologic conditions. |
| Alternative analyses | Inform feasibility analyses; identify fatal flaws | Moderate: Sufficient explorations to identify any fatal flaws and support feasibility of alternatives and establish comparative costs. |
| Final design | Inform final design analyses | High: Sufficient characterization for detailed design. |
| Construction | Confirm design assumptions | As required to verify design assumptions. |

Data obtained from information gathering and review, previous exploration phases, and preliminary analyses should be used to inform field investigation. The amount of available data should increase as the project progresses, and informed planning of targeted field investigations should improve efficiency, reduce uncertainty, and save in design and construction costs. Figure 7-12 shows a sample of a plan and profile sheet that can be used to summarize geotechnical boring data and geomorphologic mapping data collected along the levee alignment.

### Figure 7-12: Typical Plan and Profile Sheet with Geomorphology




## 3.2.3.2 Geomorphologic Mapping

Geomorphologic mapping should be used to understand historical depositional environments that should govern the locations of potentially problematic deposits and the variability of deposits. Figure 7-12 shows a sample geomorphologic plan. It can be used to target specific exploration locations if additional investigation is needed. Mapping can also be particularly useful to identify soft foundation and areas of high potential seepage when it is combined with historical performance information and geophysical surveys.

An understanding of geology and geomorphology in the project area is critical to understanding and reducing uncertainty. By understanding the potential variability of geologic units, the appropriate number of samples and laboratory tests can be selected to characterize these deposits. More variable deposits may require more sampling and testing, where less variable deposits may require less sampling and testing.

Correlation should be made between the different soil types obtained from subsurface explorations with their parental rocks by reviewing available existing geology publications and subsurface investigation information. This should help identify the formation processes that originated the deposits on which the flood risk reduction system was or will be constructed. Geological information also can be used to identify areas where additional subsurface exploration will be required, and to define the limits of the weak areas. Depositional environments should include continental (alluvial, aeolian, fluvial, lacustrine), transitional, marine, and glacial.

#### 3.2.3.3 Geophysical Surveys

Geophysical techniques can be used to obtain information on linear variations in stratigraphy, (e.g., configuration of soli/bedrock contact, foundation sands and gravels, lenses, and bar deposits). This can help assist planning of targeted explorations in areas of variation. Correlation with geomorphologic mapping can help inform uncertainty in geologic characterization of the project alignment.

Geophysical surveys also will be helpful in confirming the location of buried utilities and locating unknown utilities along the levee alignment. Additional guidance on managing utilities is presented in section 3.2.5.

As set out in more detail in EM 1110-2-1913 (USACE, 2000), geophysical investigations may also be useful in developing an understanding of existing earthen embankment levees. Understanding may be gained on issues such as:

- Changes in internal configurations of zoned embankments.
- Locations of lost or concealed metallic pipes.
- Soil gradation changes along levees.
- Embankment fracturing (including desiccation cracking), differential settlement, or subsidence.
- Possible areas of piping and internal erosion, including:
  - Piping or voids around or beneath concrete or metal structures (e.g., conduits).
  - Animal burrows and associated voids.

#### 3.2.3.4 Groundwater Conditions

Determination of the groundwater regime and its classification is important—at least at a regional level—for seepage assessments and design of cutoff walls, for informing interior drainage requirements, and for assessing the feasibility of excavating borrow material.

Classification topics for riverine situations should include:

- Whether the river in question is a gaining/losing stream and whether it acts as a groundwater divide.
- Whether there are artesian conditions present.
- Whether there is a perched water table.
- The nature of any seasonal variations in groundwater levels.

Measurement of groundwater elevations can be facilitated by piezometers, monitoring wells, relief wells, dissipation during cone penetration testing, observations from standard penetration test borings, falling head tests, and other sources.

### 3.2.4 Hazardous Materials

If the levee project formulation process identifies that mitigation of hazardous materials is required and is not the responsibility of others to address prior to levee construction, rehabilitation requirements should be included as part of the design. Whoever is responsible for the management of the hazardous waste that will be disturbed by the levee construction should work with the designer and constructor in the management, treatment, and disposal of the hazardous waste.

### 3.2.5 Utilities

The presence of overhead and buried utilities can adversely affect levee construction if not properly addressed in formulation and design. It is common for levee projects to have utility constraints, especially in urban areas. Encountering unknown utilities or required utility relocations during construction can cause significant delays and increased costs.

A survey to identify utilities in the project area should be initiated during any ground topographic survey activities during the initial planning phase (Figure 7-13). If there are known utilities, the following are practices that should be used:

- All utilities should be clearly identified in the construction documents so that the levee constructor is well informed of the utilities in the project area. This includes identifying and conveying any requirements for construction around the utilities.
- Coordination of utility relocation and levee construction activities should occur prior to and during construction between the levee owner/levee designer, levee constructor, and the utility owner in the project area.

**Figure 7-13: Example of Utilities in the Project Area**

*View of utilities (pipes and electrical lines) running along the levee landside toe right of way.*

A utility survey should be initiated during the initial formulation phase. Avoidance of existing utilities (e.g., petroleum pipelines, transmission line corridors, or large buried water transmission pipelines) may be a factor in selecting alternative levee alignments. Identification of utilities in the vicinity of the levee alignment should include research of records and field reconnaissance studies. Important information that should be gathered includes:

- Type of utility, owner, existing easement information.
- Location and depth or overhead clearance at the levee location.
- For pipelines, product carried (e.g., water, petroleum products) and risk level.
- Location and depth of water supply or drainage channels and pump stations.
- Inspection and testing reports, repair history.
- Material of construction, size, and age.

Geophysical surveys also may be conducted along the levee alignments to identify unknown utilities and other features not shown in available records or on existing as-built drawings. Identifying abandoned drainage culverts and electrical/communication conduits on projects in developed and undeveloped areas is not uncommon.

Responsibility for relocating, modifying, or removing utilities should be determined. A public utility company typically is responsible for relocating, modifying, or removing its utilities. Coordination with the various utilities should begin as soon as possible, so the work can be completed before levee construction at the utility site is scheduled to begin. The work should be reflected in the planned construction schedule for the project to track progress. Instances may occur where utility relocation may fall within the scope of design. For example, existing discharge pipelines in a levee at a municipal drainage pumping plant may be elevated in the levee, or water mains may be relocated as part of the design and construction for a levee rehabilitation project. This type of work should be coordinated between the levee owner and utility owner during the design phase.

For utilities (new or existing) that penetrate levees, there should be an analysis demonstrating that the penetration does not impact levee performance. This can be accomplished with a risk assessment. For existing utilities that may negatively impact performance, relocation or modification of the utility should be considered. EM 1110-2-2902 (USACE, 2020) provides guidance on factors to consider in evaluating penetrations through levees. The elevation of the penetration relative to the design water surface should be evaluated along with the design life of the penetration and corrosion condition.

### 3.2.6 Sources of Construction Material

Potential sources of construction material should be revisited as design progresses to confirm they will meet the specification requirements for strength, grading, and permeability. Construction materials may include earthfill, clay, sand, aggregates, riprap and other erosion protection materials, such as concrete, structural steel, sheet piling, and bentonite. This information will also support preparation of the environmental documentation (e.g., truck traffic, air quality impacts), cost estimating, and schedule preparation. See **Chapter 8** for discussion on borrow areas for earthfill for levee construction.

## 3.3 Analysis and Interpretation

The dimensions, composition, and condition of any existing levee features should be established. Known performance of these features under load should be evaluated.

### 3.3.1 Geometry

The selection of levee alignment and crest elevation (with their implications for cross-section geometry) is discussed in **Chapter 6**. An understanding of the existing topography is required to establish design geometry, including along the length of the alignment and laterally beyond the alignment at least 150 feet toward both the waterside and landside. (Note that EM 1110-2-1913 (USACE, 2000) recommends expanding this 150-foot corridor on either side of the levee in order to meet the level of accuracy required for best practice in seepage analyses. For example, a 25-foot-high levee would need ground information for at least 500 feet from the levee.)

Existing geometry can be interpreted by plotting of lateral (perpendicular to the alignment) cross sections at regular intervals. The interval that should be selected depends on the available data and the design phase. Closer spaced intervals should be plotted during later phases of design and when more data is available.

Cross sections should also be plotted at areas of interest, such as where existing non-levee features intersect the alignment, or where poor performance of existing levee features have been recorded. Performance of the natural coast or riverbank where new levees are being designed also may warrant additional cross sections. Obtaining additional survey information at these locations also may be appropriate to refine the cross sections.

### 3.3.2 Water Level and Wave Conditions

The evaluation of water level and wave conditions should be undertaken as part of the project formulation process. This is discussed in **Chapter 6**.

### 3.3.3 Interior Drainage Requirements

The impact of the proposed levee on internal drainage of the leveed area should be assessed. This can include a review of existing drainage plans or analysis of the existing topography to establish the natural drainage patterns. The presence or need of drainage ditches, culverts, and pump stations also should be noted. Where this interpretation cannot be completed, additional investigation may be required. Section 11 presents guidance on design of interior drainage features.

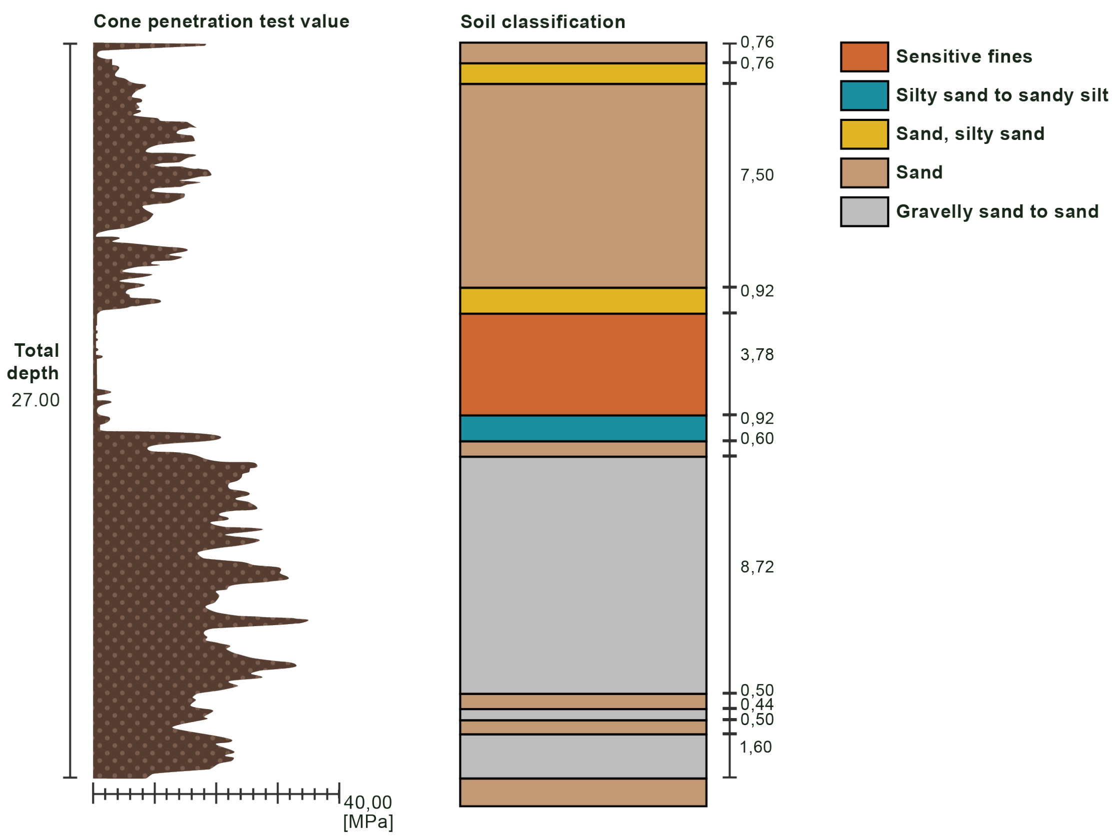
### 3.3.4 Geologic and Geomorphic Environment

Understanding the geologic and geomorphic conditions along the project alignment is critical to characterization of the existing foundation conditions and the future performance of the levee. Mapping of geologic units facilitates grouping of soils encountered in geotechnical units by similarity of depositional environments, age, structure, and mineral composition. Because testing every soil encountered during exploration is impossible, identifying similar materials for grouping is critical.

Geomorphic processes both influence how existing soils were deposited and how they may behave in the future. Rivers and coastlines are active areas with scouring, sediment transport, and deposition ongoing. The construction of levee features may affect these processes. The design team should understand these processes and the potential impacts on the expected performance of the levee features and incorporate resilience features as needed.

Figure 7-14 gives an example of an analysis of the foundation soil stratigraphy from field collected cone penetration testing.

**Figure 7-14: Typical Site Characterization Analysis**




## 3.3.5 Geotechnical Design Parameters

Geotechnical design parameters should be established for the project. These parameters should be based on engineering properties, including gradation, plasticity, compressibility, permeability/conductivity, shear strength, and density. These parameters can be established based on sampling and laboratory testing of the materials, known parameters for the mapped geologic units, and field test results. Additional guidance regarding evaluation of design parameters and performing analyses is provided in the Guidance Document for Geotechnical Analysis (California DWR, 2015).

Because exploration and laboratory test data are always limited, geotechnical parameters should be correlated to geologic and geomorphologic mapping and depositional environments. This allows estimation of the limits of conditions indicated in explorations and identification of soil units with similar engineering properties.

Longitudinal geologic profiles and geologic cross sections should be plotted to assist in interpreting geologic conditions. The geomorphologic maps should be assessed to understand differences in soil properties and stratigraphy. Although these cross sections frequently are plotted perpendicular to the levee alignment, designers should remember levees are three-dimensional, and critical conditions can occur oblique to the levee alignment and during analysis may require adjustments to reflect the three-dimensionality of a particular situation (see in particular the discussion on seepage in section 9.1.2).

### 3.3.6 Selection of Levee Reaches for Analysis

Reach selection is the process of identifying sections of the levee that possess similar characteristics. A levee reach can be represented by a single cross section and set of design parameters. Reach selection should be undertaken as part of the project formulation process, discussed in **Chapter 6**. It should be informed by the results of the risk assessment (**Chapter 4**)—where the risk is lower it may be feasible to analyze fewer cross sections and embrace the wider envelope of design parameters associated with longer reaches.

Initial reach selection may be modified during the design process based on findings from initial analyses, additional investigations, and further characterization. As set out in EM 1110-2-1913 (USACE, 2000) modifications may arise as a result of changes or clarifications in physical features and hydraulic loadings, improved geological/geotechnical/geophysical data, or further information on past performance and maintenance activities.

# 4 Levee Features

This section introduces levee system features and key design elements of those features; the design of these features is explored in greater depth in the following sections. The features are common to new levees, levee modifications, and levee rehabilitation. The levee should include features that exclude water, divert water, or control the release of water (**Chapter 2**).

The levee may be made up of multiple features and combinations of features as described in detail in **Chapter 2**. These commonly include:

- Embankment
- Floodwalls
- Closure structures
- Transitions
- Seepage control features
- Channels and floodways
- Interior drainage systems
- Pump stations
- Instrumentation

Table 7-6 shows features, associated elements, and common analyses required for design.

### Table 7-6: Levee Feature Design Requirements

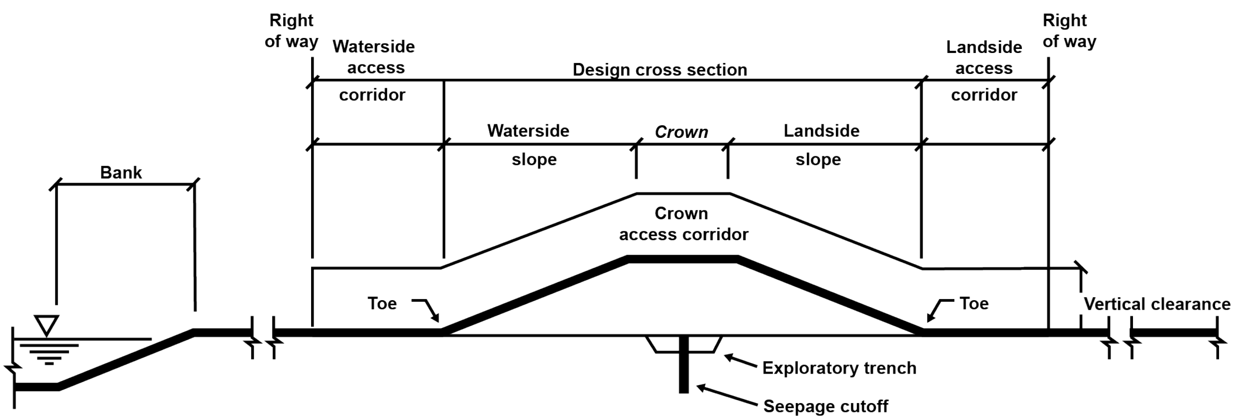
| Feature | Design Elements | Common Required Analyses |
|---|---|---|
| Embankment | • Crest elevation<br>• Geometry<br>• Exploratory trench<br>• Right of way<br>• Composition<br>• Seepage control (if needed)<br>• Stabilization measures (if needed)<br>• Erosion protection<br>• Overtopping protection (if needed) | • Seepage<br>• Stability<br>• Erosion<br>• Settlement |
| Floodwalls | • Top of wall elevation<br>• Wall type<br>• Structural materials<br>• Foundation<br>• Interface with embankment<br>• Seepage cutoff<br>• Erosion protection<br>• Penetrations<br>• Scour protection | • Seepage<br>• Overturning and sliding<br>• Wall deflection<br>• Structural failure<br>• Settlement |
| Closure structures | • Sill elevation<br>• Materials<br>• Foundation<br>• Width (access characteristics)<br>• Operation | • Seepage<br>• Overturning and sliding<br>• Wall deflection<br>• Structural failure<br>• Settlement |
| Transitions | • Geometry<br>• Material type<br>• Erosion protection | • Erosion<br>• Cracking<br>• Settlement |
| Seepage control features | • Dimensions<br>• Composition<br>• Capacity<br>• Collection | • Underseepage<br>• Throughseepage |
| Channels and floodways | • Capacity<br>• Side walls/slopes<br>• Structural elements | • Hydraulics<br>• Structural failure<br>• Global stability<br>• Flow |
| Interior drainage systems | • Size<br>• Materials<br>• Seepage protection | • Required flow capacity<br>• Internal drainage<br>• Structural failure |
| Pump stations | • Pump sizes<br>• Electrical<br>• Security<br>• Piping<br>• Sump | • Internal drainage and uplift<br>• Power requirements<br>• Structural failure |
| Instrumentation | • Type<br>• Location<br>• Data collection | • Displacement<br>• Settlement<br>• Water level/pressure<br>• Hydraulics |

# 5 Embankment

## 5.1 Elements

Figure 7-15 shows a basic embankment cross section for new levee construction. Required dimensions typically should be established based on design analyses, using applicable design criteria, or based on applicable guidelines for the project. Note the need for landside and waterside access corridors, as discussed in **Chapter 9**.

**Figure 7-15: Typical Embankment Elements**




### 5.1.1 Geometry

The embankment geometry includes the height of the embankment, the embankment crown width, and the embankment slopes. The geometric design also may include benches, berms, access corridors, and exclusion zones for utilities.

Side slopes of embankment levees should generally not be steeper than 3 horizontal to 1 vertical, as this facilitates the use of mowers for maintaining the grass cover. Unique situations (e.g., lack of space) may call for landside slopes steeper than 3:1, but the risks to maintenance operations should be fully evaluated before adoption. Levees constructed of sand may well require side slopes of 5:1 or flatter to prevent throughseepage.

Determination of the crown width should consider constructability of the levee, access needs for O&M, width needed for haul trucks, and equipment needed for floodfighting. Crown width should generally be a minimum of 20 feet, although some applicable standards may allow a lower figure.

This geometry may need to be modified, based on site-specific conditions and other factors such as available right of way, existing use (e.g., a public road on the crown), O&M, and risk-informed design analyses.

In the case of coastal levees, analyses of wave runup and overtopping will affect the final decisions about the embankment slope and crown level. The waterside slope should be adapted to limit wave runup and thus is generally flatter than that required for fluvial levees.

### 5.1.2 Inspection Trench

An inspection trench (sometimes termed exploratory trench) should be excavated under all new levees. The purpose of this trench is to expose or intercept any undesirable near-surface foundation features not identified during design. Inspection of the trench also allows the designer to assess the near-surface foundation conditions directly beneath the levee for comparison with anticipated geotechnical conditions, determined from the project's subsurface explorations to determine any areas of large unacceptable fills or utility problems.

The trench should be at or near the centerline of the levee fill, or at or near the waterside toe of sand levees, so as to connect with waterside impervious facings. The trench typically should be a minimum of 6 feet deep, measured from grade after clearing, grubbing, and stripping the levee foundation. The bottom width of the trench should be 8 to 12 feet, to allow inspection by the designer, and for subsequent backfill compaction using mechanical equipment. The trench may be deepened if local utilities are installed deeper, if the designer requires over-excavation or other treatment, and if pockets of unsuitable material are encountered during inspection of the trench. Figure 7-16 gives examples of inspection trenches.

Trenches should be backfilled with compacted fill, consistent in quality with the material to be used in the overlying embankment. Procedures for backfilling the trench to grade should be provided in the technical specifications. Where the levee design incorporates a seepage cutoff wall into the foundation, inspection of the trench excavated for installation of the cutoff wall might fulfill the purpose of an inspection trench; however, the trench should be excavated and inspected early in the construction process to provide an early warning of problems.

**Figure 7-16: Example Inspection Trench**

*An inspection trench being excavated and a view of the trench revealing the foundation soils. The trench is under a 1,800-foot-setback levee along the right bank of the Sacramento River in California.*

### 5.1.3 Materials

The embankment may be homogeneous (Figure 7-17), constructed using one soil type, or zoned (Figure 7-17), constructed using several different soil types placed in well-defined zones within the embankment.

**Figure 7-17: Typical Homogenous Embankment Section**

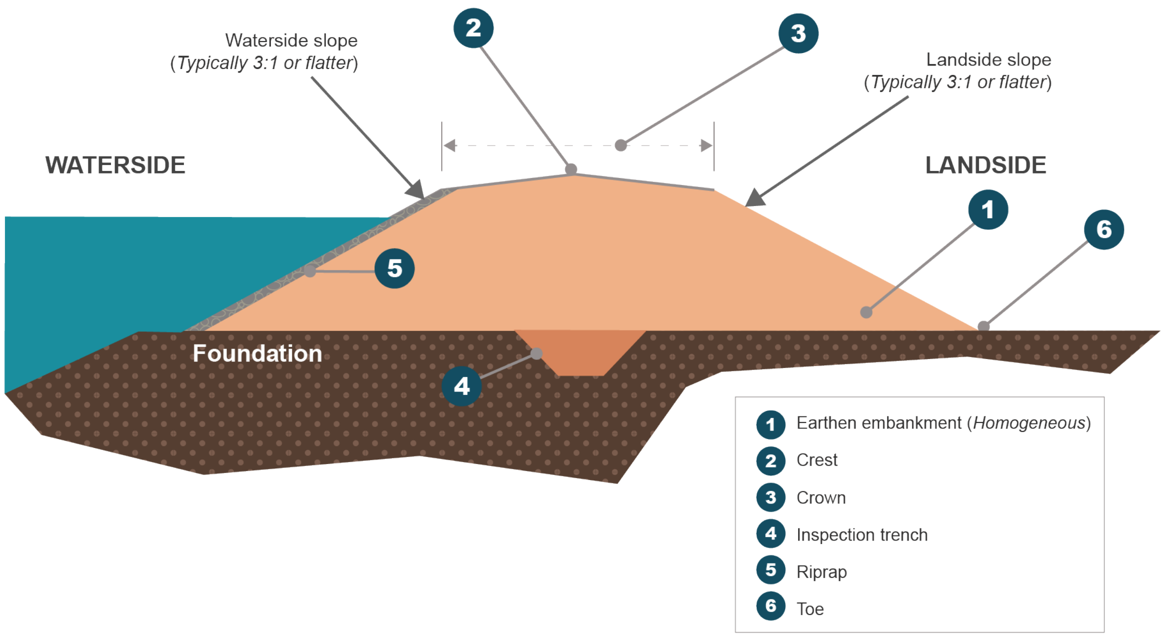
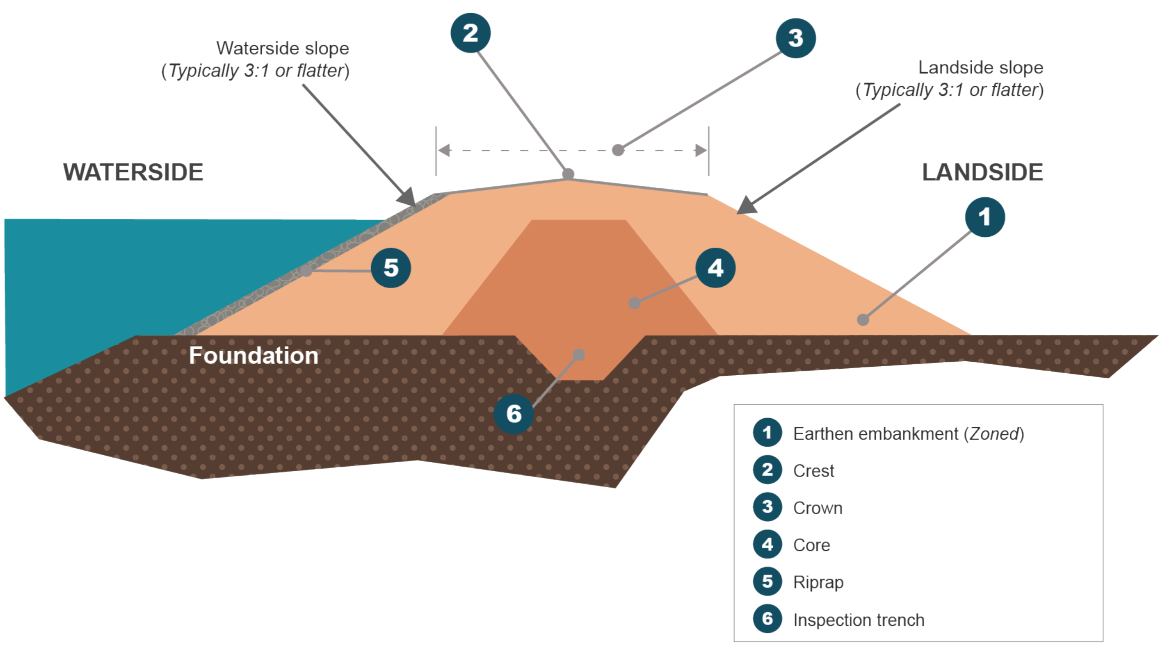



**Figure 7-18: Typical Zoned Embankment Section**



The homogeneous embankment is constructed using one soil type. The soil may come directly from borrow sources, or may require blending of materials from one or more sources to meet the strength and permeability requirements.

Where low permeability material is in limited supply, or cost-prohibited to obtain, a zoned embankment can be considered. The low permeability material is typically placed in a central core zone that is flanked on both sides with 'shell' zones formed of higher permeability soils suitable for embankment construction as described below. Note that the central core zone may be shifted to the waterside, but the required top elevation of the core for seepage control should not be reduced. Moving the core zone in the landside direction is not recommended. A low permeability blanket zone placed at the waterside face of the levee might be considered, but this approach is typically used as a throughseepage remediation for an existing levee, not for a new embankment levee.

Higher permeability material in the shell zones may improve landside embankment and foundation seepage control (section 9) and slope stability (section 5.2) by lowering the hydraulic gradient through the shell. More permeable material on the waterside slope can reduce the potential impacts of rapid drawdown (section 5.2) depending on its gradation.

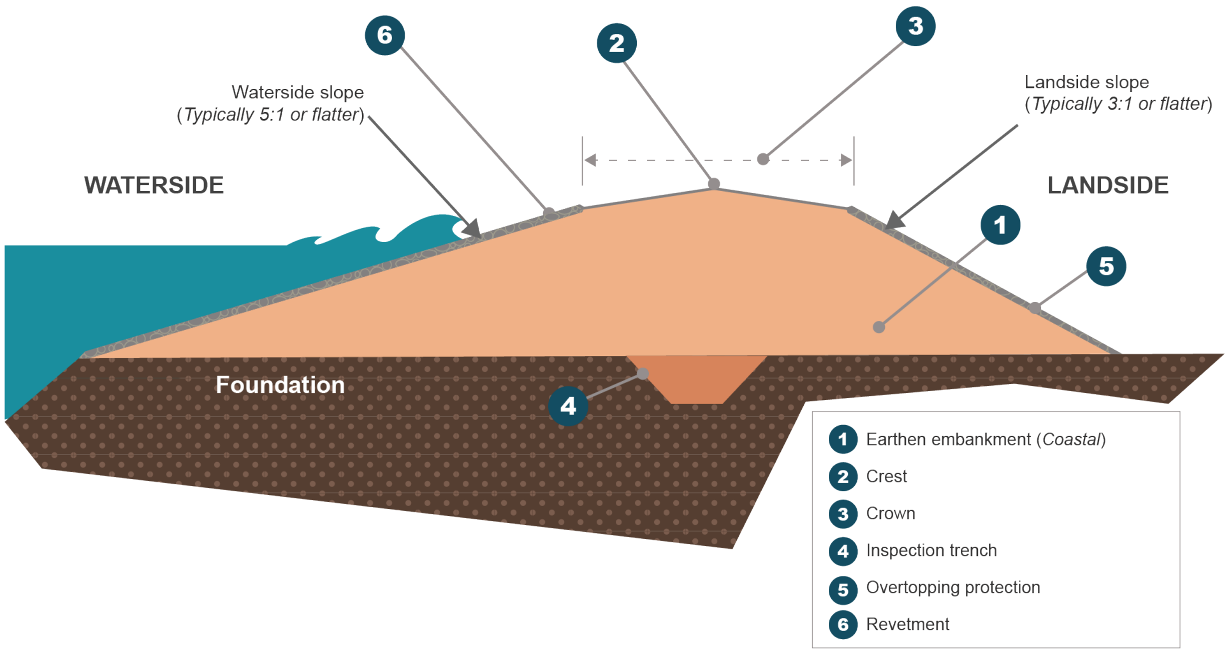
To control internal erosion, the design of a zoned embankment should also evaluate filter compatibility between adjacent zones and where zones contact the foundation. (See EM 1110-2-1913 (USACE, 2000) for further information on filter criteria and procedures for evaluating and designing filters for soil compatibility.) Incompatibility of soils can lead to soil migrating from one zone into another (piping or internal erosion), resulting in the creation of voids and possible levee failure. If filtering between zones is necessary, the following may be considered:

- Provide graded granular filters at zone contacts and on the foundation on the landside of the core zone only. Depending on soil gradations in the adjacent zones, a multi-stage filter may be needed to prevent piping. Use of geotextiles as an alternative to graded granular filters is not recommended because of the increased risk of clogging, the risk of creation of voids to one side of the geotextile, and the risk of creating preferred paths for sediment movement at joints.
- Provide one or more transition soil zones within the shell material both waterward and landward of the core with gradations that meet the filtering criteria.
- Place the shell material on the waterward and landward sides of the core so that finer, compatible material is against the core zone, but the material progressively becomes coarser toward the waterward and landward slopes.

Note that the primary consideration for the shell zone is filter compatibility to prevent internal erosion, not seepage control. The use of graded granular filters as both a filter and drain to meet seepage or stability criteria is described in section 9.

Figure 7-19 shows a coastal levee embankment and some added components that may be incorporated due to coastal hazards. The embankment may be homogeneous or zoned as described above. Surface erosion protection (e.g., rock armor) may be required to resist erosion due to wave action on the waterside slope. Due to potential for wave overtopping, it is possible that erosion protection may also be needed on the crown and landside slope.

**Figure 7-19: Typical Coastal Embankment Section**



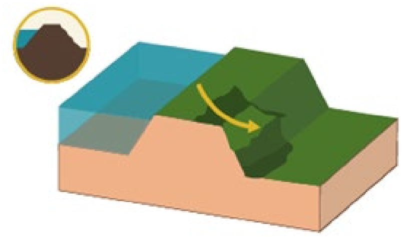

The zoned or homogenous embankment should be composed of compacted soil meeting the seepage control and strength properties established by risk-informed design analyses. The material should not include high-plasticity soils, organics, or other swelling or compressible soil. The soil should also be free from hazardous waste and environmental contaminants. The homogeneous or core zone material should be of low-erosive potential to reduce the risk of throughseepage-induced internal erosion. In terms of the shell zones in zoned embankments, in addition to the gradation requirements discussed above, the design should specify maximum heights of material layers, their moisture content, and compaction requirements; these requirements for the shell zones will likely differ from those specified for the core zone.

### 5.1.4 Common Required Analyses for Embankment Levees

The embankment should be designed and constructed to function under the required flood loading without loss of its structural integrity and stability, considering all potential failure mechanisms that could compromise its ability to function as designed.

Analyses generally completed to support earthen embankment feature and element designs include seepage (discussed in section 9.1) and resultant internal erosion, slope stability, external erosion, and settlement (which are discussed in this section). These analyses typically are deterministic but should be risk-informed and include analysis of probable failure modes. The results should be compared against established criteria that may be project-specific or regulatory. Design modifications may be required where analyses results do not indicate expected performance meeting the project objectives. This may include changing the embankment geometry or composition, or the addition of seepage control, stability, or erosion control features. These features are described next.

## 5.2 Stability Control




Instability of levee slopes is a potential failure mode that should be mitigated because it can lead to inundation of the leveed area. Instability can result from throughseepage, saturation of soft embankment soils, soft foundation soils, or undermining by erosion. The potential for instability will be affected by the following:

- Shear strengths of the levee embankment and foundation, which may vary over time.
- Pore water pressures in the soil, which likely will vary over time.
- Weight of the levee embankment and foundation.
- Compressibility of the levee foundation.
- Geometry of the levee and adjacent ground surface, which may vary over time, especially in areas vulnerable to erosion.

Waterside slope stability with rapid drawdown may also be a risk factor for the levee that should be evaluated. The starting water surface elevation for waterside rapid drawdown analysis should be the design water surface elevation. A lower elevation can be selected if the stratigraphy of the levee embankment is configured so this lower starting point will result in a more critical analysis. The drop in water surface should be selected based on historical hydrograph records for the study area. Further guidance for analyzing the waterside drawdown case is presented in the Guidance Document for Geotechnical Analysis (California DWR, 2015).

Risk reduction measures for levee slope instability can include flattening levee slopes, embankment or foundation drainage including drained stability berms, removing and replacing soft foundation or levee materials, and ground improvement measures. Design elements for stability control features are shown in Table 7-7.

**Table 7-7: Stability Feature Design Requirements**

| Stability Feature/Action | Associated Potential Failure Modes | Design Elements | Advantages | Disadvantages |
|---|---|---|---|---|
| Drainage including internal and/or foundation drainage, drained stability berm | • Slope stability<br>• Internal erosion due to throughseepage | • Alignment<br>• Width<br>• Height<br>• Composition | • Cost<br>• Lower construction risk | • Does not reduce underseepage risk.<br>• May still allow boils and require floodfighting. |
| Removing and replacing weak materials | • Slope stability<br>• Throughseepage<br>• Settlement | • Material to be removed<br>• Composition of replacement material | • Cost<br>• Lower construction risk | • Feasibility of removal.<br>• May require temporary flood risk reduction measures. |

| Stability Feature/Action | Associated Potential Failure Modes | Design Elements | Advantages | Disadvantages |
|---|---|---|---|---|
| Ground improvement | • Slope stability<br>• Settlement | • Composition of materials | • Lower construction risk | • Cost<br>• Schedule |

Flattening embankment slopes usually will increase the stability of an embankment, especially against shallow failures that take place entirely within the embankment. Flattening slopes also spreads the embankment load more uniformly and increases the length of potential slip surfaces, thereby increasing resistance to sliding, especially for deeper failure surfaces.

Slope flattening typically is considered in design of a new levee and as a possible rehabilitation for an existing levee with poor performance history caused by low strength of the embankment and foundation soils. Where stability risk includes other factors such as throughseepage, slope flattening by itself may not be an option. Drained stability berms may be more appropriate.

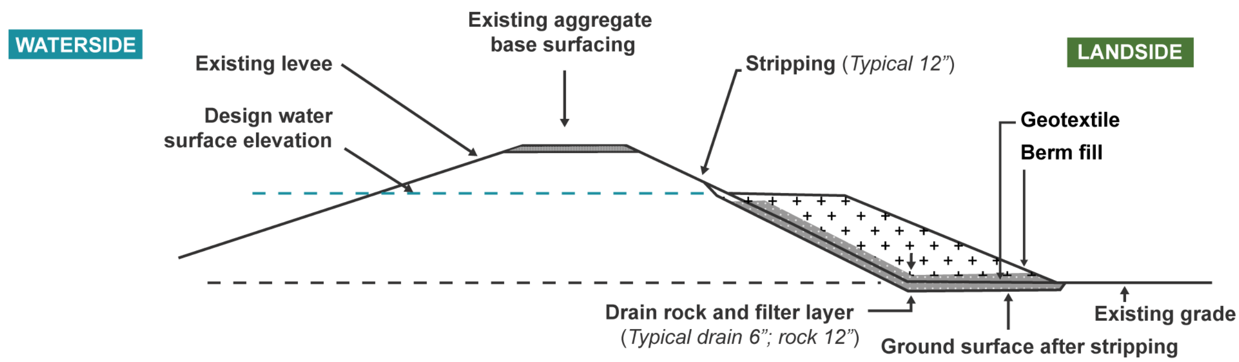
**Stability berms** increase the resisting mass at the toe, reducing the likelihood of slope instability. Table 7-8 summarizes stability berm elements and advantages and disadvantages. Design elements are discussed in the following paragraphs.

**Table 7-8: Stability Berm Elements**

| Stability Feature | Associated Potential Failure Modes | Design Elements | Advantages | Disadvantages |
|---|---|---|---|---|
| Stability berm | • Slope stability<br>• Internal erosion piping | • Alignment<br>• Width<br>• Height<br>• Composition | • Cost<br>• Lower construction risk | • May not reduce seepage.<br>• Does not reduce underseepage risk.<br>• May still allow boils and require floodfighting. |

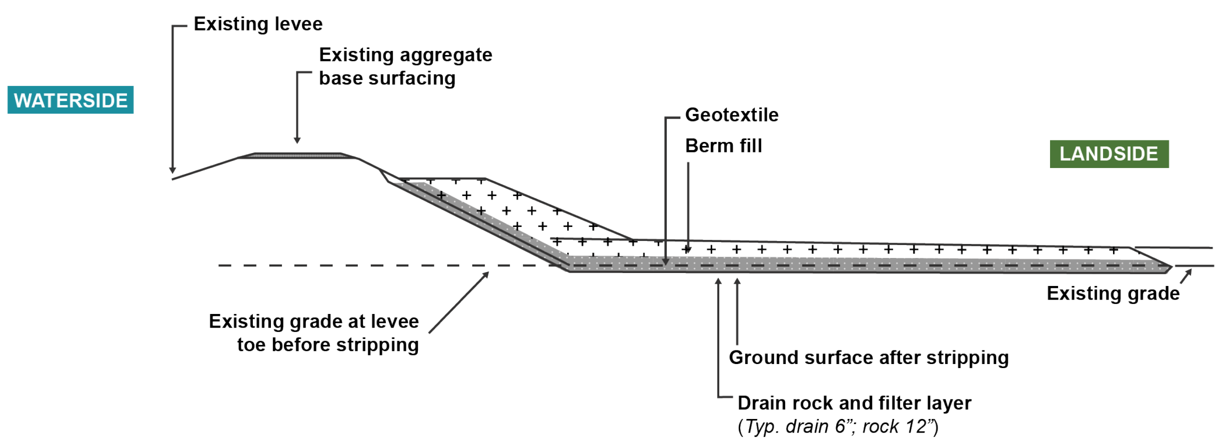
Figure 7-20 shows details for a typical drained stability berm. The berm is constructed along the landside of an existing levee. The stability berm may incorporate a drain layer on the foundation and levee slope to accommodate potential throughseepage. If the concern is only slope stability because of the low strength of the embankment soils—and throughseepage is not a concern—the drain layers may not be needed but a geotextile may still be used to provide additional strength to the berm (rather than to provide a filtration function, which is prohibited by some regulating agency guidelines).

### Figure 7-20: Typical Drained Stability Berm




Where both throughseepage and underseepage conditions exists, a combination of drained stability berm and **seepage berm** may be used to remediate slope stability and seepage. Figure 7-21 shows the details of the combination berm.

### Figure 7-21: Typical Combination Berm




The top width of the stability berm should be determined from slope stability analyses, considering actual soil properties and seepage conditions. A typical width is 15 feet. Determining the width of the seepage berm is discussed in section 9.3.2. The height of the stability berm on the landside should match the design water surface elevation on the waterside. Berm fill can be levee fill or other suitable random fill, excluding highly plastic clays or organics. The presence of locally available borrow materials should be considered in design.

## 5.2.1 Loading Conditions

The slope stability of a levee embankment usually is analyzed for the most critical loading conditions that may occur during the life of the project. These loading conditions are as follows:

- **Case 1, steady-state seepage (landside)**: Flood loading applies when water levels on the waterside exceed the landside levee toe elevation. Water exists long enough that the phreatic surface within the levee embankment has been fully established.

- **Case 2, rapid drawdown (waterside)**: The pore pressures within the levee embankment are dissipated slower than the water level is drawn down. Phreatic surfaces before and after drawdown should be defined.
- **Case 3, end of construction (landside and waterside)**: This case represents undrained conditions for low-hydraulic conductivity embankment and/or foundation soils, where excess positive pore water pressure is present because the soil has not had time to drain since being loaded in compression and shear. The phreatic surface usually is at or below the landside toe for this case.
- **Case 4, undrained loading**: It is also reasonable to perform a short-term stability analysis with the design flood loading. For many levees, this case will result in a similar slope factor of safety as the end of construction case. However, there are other situations (e.g., where there is a geotextile on soft soils or there are I-walls on the earthen embankment), the water loading can result in a lower factor of safety.

### 5.2.2 Shear Strength Selection

A range of methods may be used for selecting and assigning shear strength properties to levee embankment and foundation materials. Detailed shear strength characterization is described in Appendix D of EM 1110-2-1902 (USACE, 2003). The methods range from estimating strengths using empirical relationships (related to simple index testing) to comprehensive in situ (standard penetration testing N-values or cone penetration testing tip resistance) and detailed laboratory shear strength testing, combined with careful evaluations of the full range of soil behavior over the range of potential loadings. Published relationships may be and often are used for preliminary analyses, but advanced design and risk analysis projects may warrant site-specific testing.

In selecting shear strengths, the designer should distinguish between free-draining materials and non-free draining materials. Free-draining materials are defined as coarse-grained materials with little or no fines (typically less than 12%), so when sheared, excess pore pressures are rapidly dissipated and thus are unlikely to cause problems. Free-draining materials are assumed to remain drained, and their shear strength is characterized by their drained strength parameters for all loading conditions. Non-free-draining materials are defined as fine-grained materials or coarse-grained materials with significant fines, so when sheared, they generate (and sustain with respect to loading) excess pore pressures.

Shear strengths for analysis of specific situations should be guided by the following:

- Steady-state (case 1) and rapid drawdown (case 2): To evaluate strength and stability at steady-state and rapid drawdown, consolidated undrained triaxial with pore pressure measurement and consolidated drained triaxial, or direct shear, are performed to measure the shear strength. For long-term stability and stability during rapid drawdown, the soil may be fully or only partially saturated. However, if the soil is below the groundwater table or beneath the phreatic surface, the pore water pressures are positive, and for design purposes, the soil is assumed to be saturated. If the soil is above the water table or in a zone of capillarity and where pore pressures are negative, the beneficial effects of negative pore water pressures are conservatively neglected by assuming the pore water pressures are zero. Conventional effective stress shear strength parameters are used for both the saturated (positive pressure) and partially saturated (zero pressure) zones. The effective stress shear strength parameters are measured on specimens fully saturated before laboratory testing, regardless of the saturation that may exist in the field.

- End of construction (case 3) and undrained response to flood load (case 4): To evaluate strength and stability in these cases, unconsolidated-undrained shear tests are performed to measure the shear strength. In this case, the shear strengths are expressed as a function of total stresses, and the approach is valid for both saturated and unsaturated soils.

### 5.2.3 Stability Analyses

Stability analysis methods are well defined and can be performed using commercially available, fully documented software. As described in section 3.3.6, critical embankment sections for analysis should be selected based on geometry, loading, and geologic conditions. The accuracy of the analysis depends on the extent and quality of subsurface data and material testing available to make the stability models. Experienced professionals should be in charge of interpreting subsurface data, defining soil stratification, and assigning properties to the soil strata.

Slope stability analysis normally adopts a limit equilibrium approach to evaluate the following:

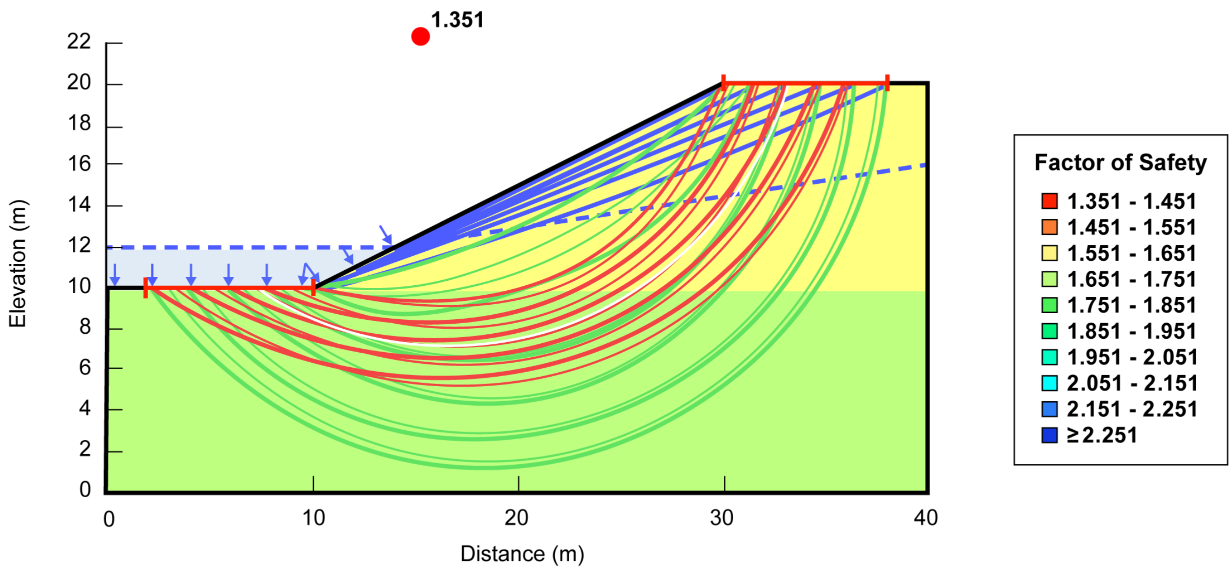
- Landside levee slope stability for static steady-state conditions corresponding to the selected water level conditions.
- Waterside levee slope stability for rapid drawdown conditions.
- Landside levee slope stability for special loading conditions.

The following is the process for a slope stability analysis:

- Select the representative levee cross section in a reach (i.e., the critical section with the least favorable condition) based on the geometric data from LiDAR, bathymetry, and stratigraphic material properties from geotechnical investigations (reach selection and cross section selection described in detail in section 3.3.6).
- Establish water surface elevations and apply appropriate surcharge loads.
- Obtain applicable pore pressures from seepage analyses (section 9.1).
- Select and verify shear strength properties of the applicable soil layers (section 5.2.2).
- Choose appropriate analysis methods.
- Perform the analysis, document the results, and compare with the past performance, if applicable.

Slip surfaces (circular, noncircular, optimization): Modern limit equilibrium method-based computer programs available for analyzing slope stability require assumption of a slip surface for which a **factor of safety** is calculated. Multiple potential surfaces are assumed and the one with the lowest factor of safety is called the most critical slip surface; the associated factor of safety for this surface should meet specified criteria. Most programs have search algorithms used to find the most critical slip surface, but the appropriateness of the resulting information should be verified prior to adoption. Figure 7-22 shows the results of a typical slope stability analysis, indicating the different slip surfaces analyzed and their associated factors of safety.

### Figure 7-22: Typical Slope Stability Analysis




When the slope stability analysis is complete, the designer should document whether the reach meets criteria or not for the applicable loading conditions with current configurations, whether a mitigation measure is needed or not.

Generally, the minimum required factor of safety is 1.4 to 1.5 for steady-state seepage (case 1), depending on water level, 1.0 to 1.2 for rapid drawdown (case 2), depending on the duration of waterside levels before drawdown and 1.3 for end of construction (case 3), and for undrained response to flood loading (case 4). Further guidance is available in EM 1110-2-1913 (USACE, 2000).

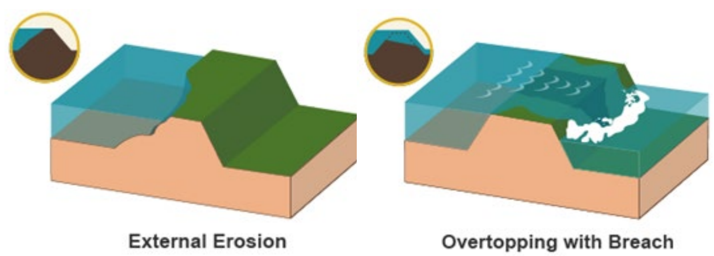
## FACTORS OF SAFETY IN MORGANZA TO GULF LEVEE PROJECT

Morganza to the Gulf is a large levee project along the Mississippi River in Louisiana. The project includes 98 miles of levees that will reduce risk to 52,000 structures and 200,000 people from hurricanes. Following Hurricane Katrina, design standards for levees and floodwalls for the levees in New Orleans were changed to increase the global slope stability factor of safety for still water scenarios from 1.3 to 1.5. These higher factor of safety requirements were applied to the feasibility study for the Morganza to the Gulf project and resulted in a significant increase in the estimated project cost.

A risk assessment was performed in 2012 to evaluate the Morganza to the Gulf levee design and also revisited the performance of levees during the Katrina event. The majority of issues during Katrina were associated with floodwalls, not embankment levees. The embankment levees that were designed to a factor of safety of 1.3 performed well. Based on the risk assessment and associated deterministic analyses, lowering the global slope stability factor of safety from 1.5 down to 1.3 did not adversely impact reliability of the levee system. The ability to learn from previously load-tested levees during Hurricane Katrina did influence this decision.

The project was approved using the reduced global factor of safety of 1.3 that resulted in a smaller levee prism and is projected to save taxpayers about $7 billion.

## 5.3 Erosion Control Features




Erosion protection can be required for different potential erosion sources, including surface runoff during precipitation, riverine or coastal flow, waves, and overtopping.

Where erosion of the landward slope occurs due to overtopping, the failure may be via surface erosion (progressive removal of surface layers) or via head cut, where the erosion causes progressive removal of vertical cuts from the landward face of the levee. The factor that determines which of these two mechanisms occurs depends on the composition of the embankment. Erosion can occur slowly or very rapidly, depending on the site conditions and the erodibility of the levee material. Either way, the process eventually leads to collapse of the levee crown. Erosion can also decrease slope stability and increase the potential for backwards erosion piping.

Erosion on the landside embankment face also can occur because of throughseepage. Erosion can result in progressive loss of embankment that shortens seepage paths and creates slope instability that can result in levee failure.

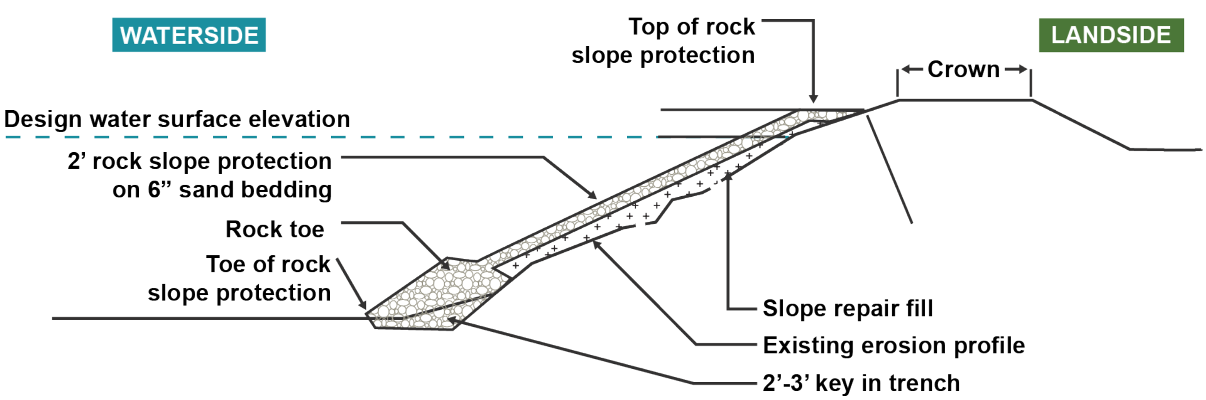
Progressive erosion of the waterside bank and toe because of scour may occur, particularly on the outer side of river bends and on coasts subject to wave action. Such scour can be particularly hazardous as it may not be observable if submerged. Bathymetric surveys should be part of site characterization and may require repeating in areas susceptible to waterside bank and toe erosion, to monitor for erosion undermining the levee. In addition, the geomorphologic process should be studied, understood, and monitored to identify locations potentially susceptible to scour. Table 7-9 summarizes erosion control feature design elements and their design requirements.

### Table 7-9: Erosion Control Feature Design Requirements

| Erosion Control Feature | Associated Potential Failure Modes | Design Elements | Advantages | Disadvantages |
|---|---|---|---|---|
| Armoring/ bedding | • Surface erosion<br>• Overtopping erosion<br>• Riverine erosion<br>• Wave erosion | • Areal extents<br>• Height<br>• Composition | • Low maintenance | • Aesthetics<br>• Cost<br>• Constructability |
| Vegetation | • Surface erosion<br>• Overtopping erosion<br>• Riverine erosion<br>• Wave erosion | • Areal extents<br>• Height<br>• Composition | • Cost<br>• Aesthetics | • Increased maintenance |

Erosion likelihood is commonly mitigated through armoring and vegetating the levee. Figure 7-22 shows typical rock armoring details for the waterside face. Design elements for armoring include rock gradation, locations, height, and placement techniques.

### Figure 7-23: Typical Erosion Repair




Vegetation is most commonly used as protection against surface runoff erosion of the embankment slopes. Vegetation should be designed based on local conditions and regulations, as described in section 2.3.5.

**Resilience** to overtopping can be increased by armoring the crown and rear face. This is particularly recommended at locations of controlled overtopping, as discussed in section 10.2.

### 5.3.1 Erosion Analysis

Erosion analyses can include simple, empirical review of bank or coastline erosion and expert elicitation of likely morphological processes.

At specific locations, evaluation of soil erosion due to currents, waterside wind/waves, and landside overtopping can be used via the widely used linear excess stress erosion models to estimate erosion rate as a function of hydraulic shear stress and soil erosion resistance.

Riprap armoring/bedding can be sized using widely used stability relationships for rock of different sizes, taking account of the fact that well-interlocked permanently placed rock will be more stable than loose launchable rock toes designed to move into and fill areas of scour (see EM 1110-2-1601 (USACE, 1994b)). Safety factors may need to be increased to take account the risk of freeze-thaw and of vandalism. (A minimum individual rock weight of 80 pounds is usually sufficient to prevent theft and vandalism.)

SOURCES FOR DETAILED GUIDANCE FOR PERFORMING EROSION ANALYSIS

- EM 1110-2-1913 Design and Construction of Levees (USACE, 2000).
- EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002).
- Guidance Document for Geotechnical Analysis (California DWR, 2015).
- *Evaluation of Potential for Erosion in Levees and Levee Foundations*, Center for Geotechnical Practice and Research #64 (Duncan *et al.*, 2011).
- *International Levee Handbook* (Eau and Fleuves, 2017).

## 5.4 Settlement Control

Levees often are constructed over areas with highly variable subsurface conditions. Although it is desirable to construct levees in foundation conditions that require minimum post-construction measures to account for settlement because of alignment constraints, it often becomes necessary to construct levees across highly compressible foundations. Table 7-10 expands on the procedures commonly applied to levee projects to deal with this situation.

**Table 7-10: Settlement Design Procedures**

| Settlement Procedure | Design Considerations |
|---|---|
| Remove and replace | • May be used to reduce settlement in areas where shallow soft deposits or fill layers exist.<br>• Becomes less feasible where compressible layers are deep, or where high-water tables exist that will require dewatering during construction. |
| Staged construction | • Requires adequate time for consolidation.<br>• Settlement monitoring instrumentation readings during construction may be used.<br>• Levee lifts can be scheduled to be placed after completion of the original construction. |
| Prefabricated vertical wick drains | • Allow rapid construction of levees over very soft foundations.<br>• Design is optimized by maximizing the wick drain spacing to achieve an appropriate degree of consolidation needed within the time available for the consolidation. |

| Settlement Procedure | Design Considerations |
|---|---|
| | • The long-term performance of wick drains should be considered in seepage evaluations. |
| Preloading and surcharge fills | • Typically uses material not meeting levee fill requirements; it is placed before levee construction and removed before final levee construction.<br>• Where stability conditions allow, surcharge placed to heights in excess of the final levee height may be placed to accelerate the consolidation time needed during construction. |
| Soil improvement or amendment | • Soft foundation soils can be excavated, treated (such as drying), and replaced in lifts and compacted.<br>• Additives used in soil improvement should have the effects on hydraulic conductivity and strength evaluated and measures taken to avoid any negative impact.<br>• Deep mixing methods are viable alternatives with considerations:<br>- Deep mixing methods introduce hardened elements in the levee and/or levee foundation that can cause differential settlement.<br>- Projects that are candidates for deep mixing methods are those where levee materials can deform without cracking, rather than those that are stiff or hard. |

### 5.4.1 Settlement Analysis

Settlement analysis methods are well-defined and can be performed using commercially available, fully documented software. The accuracy of the analysis depends on the extent and quality of subsurface data and material testing available to make the stability models. Guidance on performing settlement/stability analyses for levees is presented in EM 1110-1-1904 (USACE, 1990).

## 5.5 Modification and Rehabilitation

The following sections provide guidance for some typical modifications and/or rehabilitation that may be required for existing embankment levees to implement risk reduction measures. Such measures may be needed to accommodate settlement or water surface elevation design criteria changes for riverine levees because of changed hydrology, or coastal levees because of hurricane tides and storm surges.

### 5.5.1 Seepage and Stability

Seepage analysis is discussed in section 9.1. Required stability modifications or rehabilitation can be made by constructing one or more of the options discussed in the remainder of section 9: cutoff walls, seepage and stability berms, blanket drains, and relief wells.

### 5.5.2 Levee Crest Elevation Raise

Levee enlargement by adding embankment fill or constructing a floodwall on the levee crown are the two most economical and practical approaches to provide additional levee height.

### 5.5.2.1 Enlargement Using Earthwork

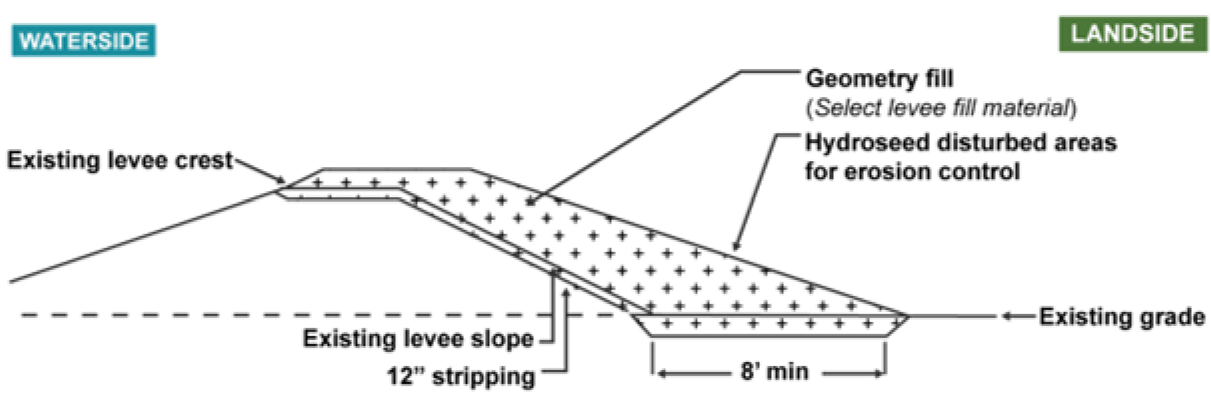
Levee enlargement may be accomplished using one of the following three methods:

- Landside enlargement by elevating the levee crown and thickening the landside slope using suitable compacted fill, as shown in Figure 7-24, with a maximum slope of 2 horizontal to 1 vertical for the bench cut into the existing levee. Analysis of landside material placement should take material compatibility and filter requirements into account.
- Waterside enlargement by elevating the levee crown and thickening the waterside slope using suitable compacted fill.
- Straddle enlargement by elevating the levee crown and thickening the waterside and landside slopes using suitable compacted fill.

The advantages and disadvantages to be considered for each method may include:

- Methods with landside slope enlargement could require additional right of way.
- Methods with waterside slope enlargement could be more costly if rock slope protection is present, or if groundwater or tidal conditions exist.
- Methods with waterside slope enlargement may have more environmental impacts, may change the erosion pattern within the river, and can encroach on the hydraulic capacity of the channel, possibly increasing the design water level for all levees in the system.

**Figure 7-24: Typical Embankment Raise**




Geotechnical investigation should be completed to confirm the material properties of the existing embankment soil and proposed fill. New fill should be comparable or better than the existing fill; compacted to at least the same density. Enlargement for stability and settlement should be checked, and if applicable, throughseepage or underseepage should be assessed, following the guidelines presented in this chapter. After stripping, the new fill should be bonded with the existing fill by scarifying and compacting existing surfaces and benching existing slope surfaces. Thin fills should be avoided. The horizontal width of new fill should be wide enough to accommodate hauling and compaction equipment.

#### 5.5.2.2 Raising Embankments Using Floodwalls

Floodwalls (section 6) can be constructed on levee earthen embankments to increase crest elevation. This method may be appropriate when the existing right of way is not available or is too expensive to acquire, if the foundation conditions do not permit an increase in the levee section using earthwork, or where short wall height is needed to increase the levee crest elevation.

Floodwall advantages and disadvantages are:

- This may not be economical compared to using earthwork for the enlargement, and economic comparisons should be performed.
- The floodwall would restrict access to the waterside of the levee.
- This may affect O&M activities and access to the waterside slope for emergency response activities.

The floodwall should have adequate stability to resist all forces that may act on it. Geotechnical investigation of the existing embankment should be considered to confirm the material properties used to evaluate floodwall stability. Two common types of a floodwall used for enlargement are the I-wall and the inverted T-wall, as described in section 6.

Note that I-walls are less robust than other wall types, particularly in soft soils and when the wall is taller than a few feet. They can be more susceptible to overtopping erosion in certain conditions.

## 5.6 Seismic Considerations

Levee seismic performance generally is of moderate concern because of the low probability of a damaging seismic event, especially in combination with a flood event. However, regions of high seismicity may warrant a review of levee performance. In such high seismic areas, a levee may be evaluated for the likelihood of foundation failure because of liquefaction for a design seismic event that may result in slope failures and loss of freeboard. This may require geotechnical evaluations of the levee and foundation to better characterize the materials present. If liquefaction is not determined to be an issue, further evaluation may not be needed. However, if liquefaction may be an issue, further evaluation should be considered, including a risk assessment of failure and impacts on the leveed area. Guidance for seismic evaluations of levees is presented in the Guidance Document for Geotechnical Analysis (California DWR, 2015).

Risk-reduction measures for seismic concerns include the removal and replacement of susceptible foundation soils, ground stabilization measures, and compaction grouting. Another risk mitigation measure is the development of a contingency plan to rebuild or partially rebuild the levee within a short timeframe if liquefaction were to occur.

# 6 Floodwalls

Different wall types may be required based on the desired height, the available right of way, geologic conditions, operability, and aesthetics. For further guidance, refer to USACE EM 1110-2-2502 (USACE, 1989a). Typical design steps for a floodwall would include:

- Establish the design flood water surface elevation or profile along the length of the barrier.
- Set the required height of the barrier by combining the design water surface elevation with estimated wave heights and freeboard allowance.
- Establish design load combinations, including dead loads, water and wave loads, wind loads on exposed surfaces, and allowance for debris impact.
- Design all of the components based on critical load combinations to meet applicable structural codes for the material used (e.g., American Institute of Steel Construction Manual for steel members).
- Evaluate the barrier-foundation system for overturning and sliding stability under the worst combination of design loads. Safety factors of 1.25 or higher should apply.

Concrete T- or L-walls generally have been found to provide greater resilience and are preferred. These walls have T- or L-shaped foundations that provide overturning and sliding resistance. In addition, the wall stiffness reduces deflections and the potential for formation of gaps between the wall and soil that can lead to wall failure.

Sheetpile walls sometimes are used when insufficient right of way can be obtained to construct concrete walls. One advantage of sheetpile walls is they can be driven or pushed into the surface to form an integrated seepage cutoff. However, sheetpile walls historically have deflected under load, leading to wall rotation and subsequent overtopping and failure. If used, detailed analyses are required to establish expected sheetpile wall deflection under the design loads.

Stabilized earth walls also can be considered. A stabilized earth wall essentially is a steepened waterside slope made stable by reinforcing. This reduces the footprint of an earthen-type structure. Erosion protection will be required to protect such walls from flow and waves.

Demountable floodwalls may be considered in situations where a permanent obstruction is undesirable, although they typically include some permanent foundation features.

Each of these types is discussed in more detail in the following sections.

## 6.1 Reinforced Concrete Floodwalls

Reinforced concrete floodwalls are an option for coastal and riverine floodplains and may be used aloneor in combination with embankment levees to provide the required level of flood risk reduction. They generally are employed where space is limited and thus an embankment levee may not be a viable option.

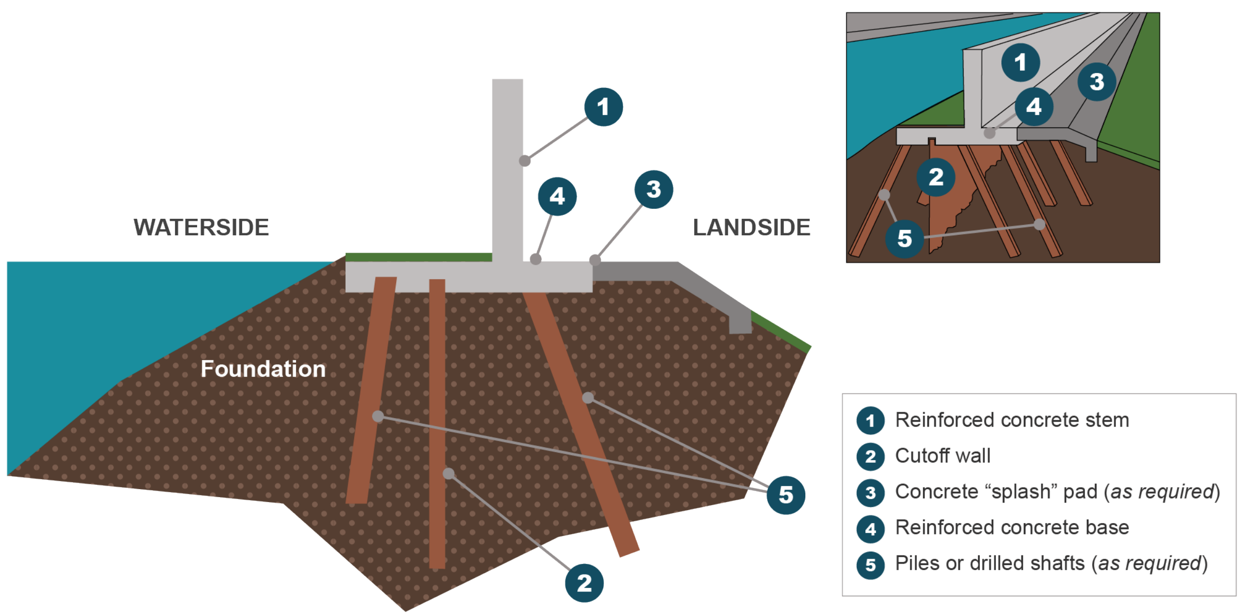
Table 7-11 and Figure 7-25 illustrate the design elements for the floodwall and the analyses normally required for design. Walls should be designed for all applicable static, hydrodynamic (wave), and resilience design loads. Boat or barge impact loads also should be considered if applicable, based on levee location. Seismic loading also may apply, but usually does not control the design of the wall.

Floodwalls may breach through different processes that cause loss of wall height or allow water to pass through the wall alignment. A wall-related failure mode generally is related to some geotechnical instability or an internal structural failure causing an uncontrolled release of water.

**Table 7-11: Floodwall Design Elements**

| Feature | Design Elements | | Common Required Analyses |
|---|---|---|---|
| Floodwalls | • Top of wall elevation<br>• Structural materials<br>• Interface with embankment<br>• Erosion protection | • Wall type<br>• Foundation<br>• Seepage cutoff<br>• Penetrations<br>• Scour protection | • Seepage<br>• Overturning and sliding<br>• Wall deflection<br>• Global stability<br>• Settlement |

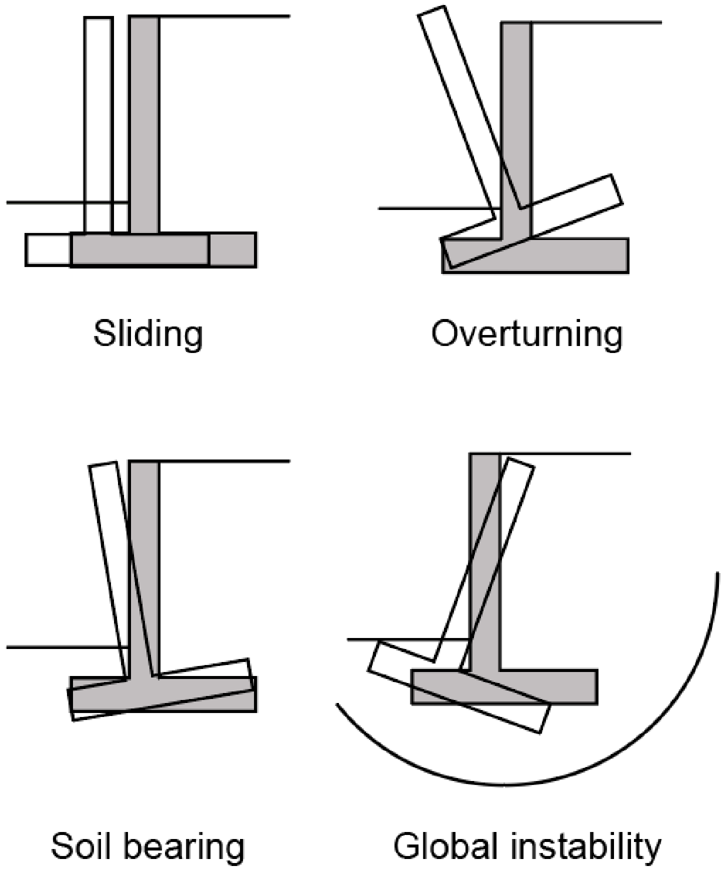
**Figure 7-25: Example Floodwall**




## 6.1.1 Geotechnical Design

A geotechnical stability failure typically causes some displacement associated with sliding, overturning, loss of bearing, or undermining of the wall. Displacement of a wall can be initiated by inadequate stability, either translational or rotational, or a combination of both. Figure 7-26 shows the potential failure modes for a floodwall. Contributing factors commonly associated with a wall instability include overtopping erosion, wave over splash erosion, overload, increased uplift, heave, low soil strength, or waterside gap formation.

#### Figure 7-26: Floodwall Geotechnical Failure Modes




Analyses generally completed to support the geotechnical design of floodwalls include:

- Wall overturning
- Wall sliding
- Wall deflection and gap formation
- Settlement
- Bearing failure
- Flotation
- Underseepage
- Slope instability undermining the wall

Additional design may be required for floodwalls that have openings to allow pipe penetrations and drainage. Seepage and piping issues are discussed separately in section 9.

#### 6.1.1.1 Gravity Foundations

Foundations are normally designed using allowable stress theory. For normal operations and design flood stages—and to avoid overturning—the resulting load on the foundation should normally remain within the middle third of the base.

Gravity foundations where the floodwall is supported on soil are generally only used for riverine or inland levees. Further, protecting the soil foundation from scour that can occur in both flowing riverine floods and the wave action in coastal hurricane floods is critical. A concrete sheetpile toe wall should be on the waterside of the slab. The toe wall should be designed for scour depths and also provide cutoff. Guidance for designing soil founded structures is available in EM 1110-2-2100 (USACE, 2005) and also EM 1110-1-1905 (USACE, 1992a).

#### 6.1.1.2 Pile Foundations

Pile foundations can be required for inland floodwalls with weak soils. Coastal floodwalls often require pile foundations to resist the high lateral forces produced by waves.

Pile foundation designs currently are performed by using either allowable stress design or load and resistance factor design, but normally use allowable stress design with service loads. Further guidance on pile foundations for levees may be found in EM 1110-2-2906 (USACE, 1991).

Pile capacity in firm soils and those that bear on rock can be designed efficiently with the end bearing greatly increasing bearing capacity. Where rock foundations are shallow, there may be insufficient tension capacity provided by pile wall friction to overcome hydrostatic uplift. As a result, anchoring tension rods into the firm foundation may be necessary.

In softer soils, ground instability and settlement can greatly increase the loading on the piles. Ground improvements should be considered to relieve the loads placed on piles.

The most economic design may be soil improvements versus piles. Those improvements include preloading, the addition of stability berms, and the use of deep mixing methods, although deep mixing methods are not always less costly.

Where concrete floodwalls are added to the crest of earthen embankment levees, the additional resistance offered by pile foundations may be required to avoid instability of the embankment (section 5). Piles are used to nail the foundation and provide the added resistance that, when combined with the soil capacity, meets the embankment stability factors of safety. However, impacts of pile driving on the existing features of the earthen embankment need to be taken in to account and any resultant deformations and cracking will need to be remediated.

If used, pile foundations can account for approximately 30% of the structure cost.

### 6.1.2 Overtopping Resilience Design

Overtopping of floodwalls can result in scour of the landside and subsequent wall failure. Coastal floodwalls and inland floodwalls within designated overtopping reaches are the most vulnerable. In these areas special care should be provided in the design of overtopping protection.

Research has established empirical relationships to permit the design of scour protection measures according to the design overtopping rates and volumes. Measures can include hard surfaces, such as concrete or riprap, grass/sod for lower rates, or other erosion control features.

### 6.1.3 Structural Design

The structural design includes design of individual wall structural members and additional checks such as:

- Flexural and shear strength of piles, wall stem, and base slab heel and toe.
- Foundation heel and toe flexural and shear.
- Wall deflection.
- Key shear strength (and flexural strength if needed).
- Rebar laps and embedment.
- Steel reinforcing ratios for all structural members (including for crack control).
- Wall connection to existing tie-in structures, such as bridge abutments, wall transitions, and others.
- Anchorage to existing structures.

Additional design may be required for floodwalls that have openings to allow pipe penetrations and drainage.

The initial step to design floodwalls is selection of the structural design criteria, which includes load combinations, factors of safety, or demand-to-capacity ratios.

- **Loading conditions**. Load cases for riverine and coastal hurricanes are similar in that hydraulic loads are the principal loads. Load factors (and allowable stress design) consider the return period of the storm event and load case combination frequency. Load combinations may be identified as usual, unusual, and extreme, as shown in Table 7-12.

## DETAILED GUIDANCE FOR STRUCTURAL DESIGN

The following documents may be referenced.

- USACE EM 1110-2-2100 (USACE, 2005) Stability Analysis of Concrete Structures.
- USACE EM 1110-2-2104 (USACE, 2016) Strength Design for Reinforced Concrete Hydraulic Structures.
- USACE EM 1110-2-2502 (USACE, 1989a) Floodwalls and Other Hydraulic Retaining Walls.
- USACE EM1110-2-2107 (USACE, 2022) Design of Hydraulic Steel Structures.
- USACE EM 1110-2-2906 (USACE, 1991) Design of Pile Foundations.
- American Concrete Institute 318-14 Building Code Requirements for Structural Concrete (ACI Committee 318, 2014).
- American Concrete Institute 350-20 Code Requirements for Environmental Concrete Structures (ACI Committee 350, 2021).
- American Association of State Highway and Transportation Officials 2012 Load and Resistance Factor Design Bridge Design Specifications (AASHTO, 2012).
- U.S. Bureau of Reclamation 2019 Best Practices in Dam and Levee Safety Risk Analysis (USACE and U.S. Department of the Interior, Bureau of Reclamation, 2019).

When selecting load combinations, see the following from the above list: EM 1110-2-2104, American Association of State Highway and Transportation Officials, and U.S. Bureau of Reclamation allowing appropriate selection of design criteria. American Society of Civil Engineers Structural Engineering Institute 7-22 (ASCE, 2022) can also be used when including conditions for flood loads.

**Table 7-12: Load Condition Categories**

| Load combination Categories | Annual Probability (p) | Return Period (tr) |
|---|---|---|
| Usual | Greater than or equal to 0.10 | Less than or equal to 10 years |
| Unusual | Less than 0.10 but greater than or equal to 0.0013 | Greater than 10 years but less than or equal to 750 years |
| Extreme | Less than 0.0013 | Greater than 750 years |

  - Usual loads refer to loads and load conditions, which are related to the primary function of a structure and can be expected to occur frequently during the service life of the structure. A usual event is a common occurrence, and the structure is expected to perform in the linearly elastic range.
  - Unusual loads refer to operating loads and load conditions of infrequent occurrence and/or short term. Since risks can be controlled by specifying the sequence or duration of activities and/or by monitoring performance, construction and maintenance loads are classified as unusual loads. Loads on temporary structures used to facilitate project construction are also classified as unusual. For an unusual event, some minor nonlinear behavior is acceptable, but any necessary repairs are expected to be minor.
  - Extreme loads refer to events that are highly improbable and can be regarded as emergency conditions. Such events may be associated with major accidents involving impacts or explosions and natural disasters because of earthquakes or flooding, which have a frequency of occurrence that greatly exceeds the economic service life of the structure. Extreme loads also may result from the combination of unusual loading events. The structure is expected to accommodate extreme loads without experiencing a catastrophic failure, although structural damage that partially impairs the operational functions are expected, and major rehabilitation or replacement of the structure may be necessary.

- **Concrete resistance**. Building Code Requirements for Structural Concrete and Commentary (ACI Committee 318, 2022)specifications dictate minimum concrete strength, maximum water cement ratios, and other durability requirements. The critical concern for mass concrete is the increased thermal stresses brought on by the hydration process. Thicker placements are more susceptible to increases in thermal stresses. Unacceptable cracking can compromise the structural integrity of the reinforced concrete. A set thickness does not exist among the codes that establishes whether the placement is mass concrete. A common threshold is 5 feet; other considerations are the ambient temperature, structure size and restraint, and concrete mix ingredients.
- **Analysis methods** predominantly use load and resistance factor design. Concrete design should consider both serviceability (durability) and strength. Durability criteria limits tension stress and the resulting tension cracks that lead to spalling and corrosion of reinforcement. (Note that the provision in Building Code Requirements for Structural

Concrete and Commentary (ACI Committee 318, 2022) that allows tension in reinforcing steel used in floodwalls to approach yield should be limited to infrequent events.)

#### 6.1.3.1 Design of Submerged Structures

Structures submerged in water and those exposed to water loadings are considered concrete hydraulic structures. The term implies serviceability (durability) is part of the design. The duration and frequency of exposure to water should be considered when selecting the design criteria. Durability is increased when crack width in concrete is minimized, resulting in less penetration of water to corrode rebar. This can be accomplished in design by limiting the tension in the reinforcement and increasing the amount of concrete cover. Alternatively, epoxy-coated rebar or stainless-steel rebar can achieve the same service life.

**FURTHER GUIDANCE FOR STRUCTURAL DESIGN OF MASS CONCRETE SEAWALLS**

The following documents cover the formulation, design, and construction of seawalls:

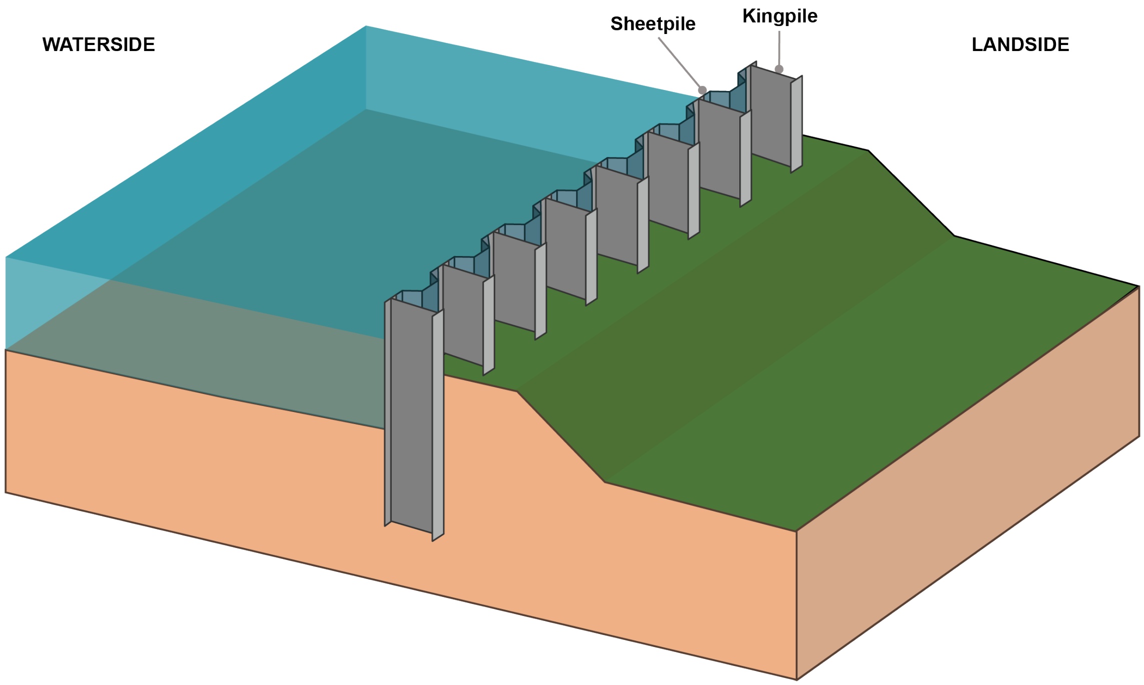
- USACE EM 1110-2-1100, Coastal Engineering Manual-Part V (USACE, 2002b).
- USACE EM 1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads (USACE, 1995a).

## 6.2 Steel Sheetpile Floodwalls

Steel is the most common material used for sheetpiling walls because of its inherent strength, stiffness, ductility, relative light weight, and long service life when protected from corrosion. Steel sheetpile walls are commonly known as I-walls and consist of a driven, vibrated, or pushed row of interlocking vertical pile segments to form a continuous wall. The wall may extend to the full height with sheetpile or be constructed with sheetpiling in the embedded depth and a monolithic cast-in-place, reinforced concrete wall in the exposed height (sheetpile with concrete cap). A possible disadvantage of I-walls can be excessive deformation, leading to poor performance and potential failure under the maximum design loads.

Combined wall systems are typically used when regular sheetpiles are not strong enough to carry the required loads. Combined wall systems (see Figure 7-27) consist of two primary components, the king pile and the intermediary sheetpiles. The intermediary sheetpiles transfer horizontal loads to the king piles, while the king piles carry the majority of the bending moment and shear, and also may carry vertical loads. The wall components are driven, vibrated, pushed, or drilled into place.

**Figure 7-27: Example Combined Sheetpile Floodwall System**




## 6.2.1 Design Procedures

The design of sheetpile floodwalls require the following components:

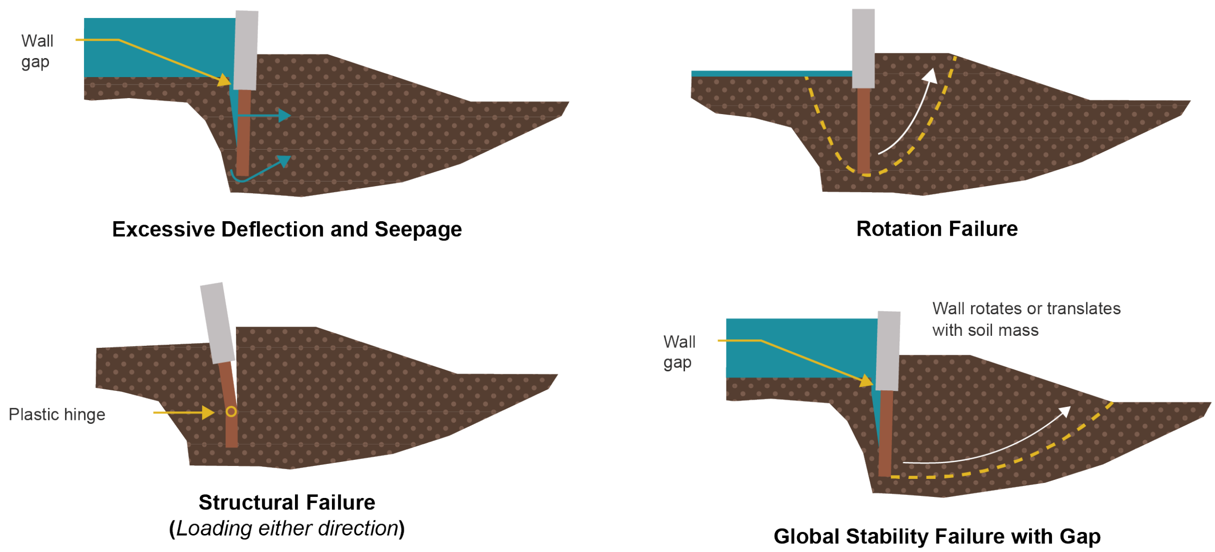
- Evaluation of the forces and lateral pressures that act on the wall.
- Determination of the required depth of piling penetration.
- Computation of the maximum bending moments in the piling.
- Computation of the stresses in the wall and selection of the appropriate piling section.
- Design of any support system.

However, before these operations can be initiated, certain preliminary information should be obtained. In particular, the controlling dimensions should be set. These include the elevation of the top of the wall, the elevation of the ground surface in front of the wall (commonly called the dredge line), the maximum water level, the mean tide level or normal waterside elevation, and the low water level. A topographical survey of the area also is helpful.

## 6.2.2 Potential Failure Modes

As shown in Figure 7-28, the potential failure modes of a steel sheetpile wall include excessive deflection and seepage, structural failure, rotational failure because of inadequate pile penetration, and global stability failure.

**Figure 7-28: Sheetpile Wall Failure Modes**




## 6.2.3 Loads

The loads on a sheetpile wall are primarily from the soil and water surrounding the wall and from other influences, such as surface surcharges and external loads applied directly to the piling, including earth pressures, water loads (i.e., hydrostatic and seepage forces), and surcharge loads and other applied loads. The loading conditions—including usual, unusual, and extreme cases—are the same as for concrete floodwalls (section 6.1). See EM 1110-2-2502 (USACE, 1989a) for further details.

## 6.2.4 Methods of Analysis

The two basic types of steel sheetpile walls are cantilevered walls and anchored walls.

A cantilever wall is assumed to rotate as a rigid body about some point in its embedded length. This assumption implies the wall is subjected to the net active pressure distribution from the top of the wall down to a point (subsequently called the “transition point”) near the point of zero displacement. The design pressure distribution then is assumed to vary linearly from the net active pressure at the transition point to the full net passive pressure at the bottom of the wall. Equilibrium of the wall requires that the sum of horizontal forces and the sum of moments of any point are both equal to zero. The two equilibrium equations may be solved for the location of the transition point and the required depth of penetration. Walls designed as cantilevers usually undergo large lateral deflections and are readily affected by scour and erosion in front of the wall. Because the lateral support for a cantilevered wall comes from passive pressure exerted on the embedded portion, penetration depths can be quite high, resulting in excessive stresses and severe yield.

An anchored sheetpile wall derives its support by two means—passive pressure on the front of the embedded portion of the wall, and anchor tie rods near the top of the piling. For higher walls, the use of high-strength steel piling, reinforced sheetpiling, relieving platforms, or additional tiers of tie rods may be necessary. The overall stability of anchored sheetpile walls and the stress in the members depends on the interaction of a number of factors, such as the relative stiffness of the piling, the depth of piling penetration, the relative compressibility and strength of the soil, and the amount of anchor yield. In general, the greater the depth of penetration, the lower the resultant flexural stresses. Design of an anchored sheetpile wall usually uses the free earth support method or fixed earth support method.

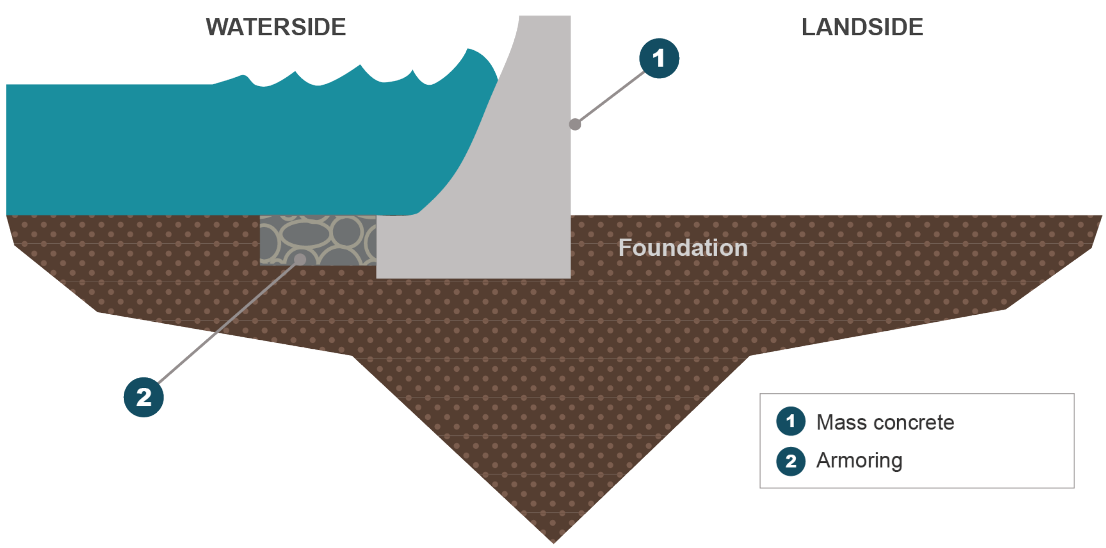
### 6.2.5 Overtopping Resilience

As discussed for other wall types (section 6.1.2), overtopping of floodwalls can result in scour of the landside and subsequent wall failure; therefore, scour protection should be provided. In the case of sheetpile walls, consideration should also be given to means of limiting penetration of floodwater down the face of the sheetpiling to a location where it may reduce shear strength.

## 6.3 Mass Concrete Gravity Walls

Mass concrete gravity walls are often used in the coastal environment because of their ability to better manage large wave forces. A gravity wall is typically a massive, concrete structure with its weight providing stability against sliding forces and overturning moments. The key functional element in design is establishing the crest elevation to minimize overtopping, whether from excess river levels or from storm surge and wave runup. Figure 7-29 shows an example of a typical mass gravity wall.

**Figure 7-29: Typical Mass Gravity Wall**




Where the wall is subject to wave attack, the front face should be curved to deflect wave runup. Under some conditions, wave runup can also be reduced by the inclusion of steps on the front face. The depth of excavation into the foundation will depend on local geotechnical conditions and embedment needed for stability. If the foundation is not suitable to support the wall, or if settlement with time is an issue, a pile foundation or other type of ground improvement may be needed. Depending on height, size, and loading, reinforcing steel may be required for waterside and landside faces, and for the base if piles are used. Proper closure of the wall at the ends either to existing topography or to other flood risk-reduction features is an important consideration.

Waterside toe protection (armoring) is typically required to prevent undermining from wave action.

Finished grading requirements for land development may require some backfilling on the landside of the wall. Maintenance roads may also be present. These loads should be considered in designing the wall. Drainage penetrations through walls may be needed for gravity drainage of landside areas when waterside conditions allow, or for drainage pumping station discharges.

### 6.3.1 Design Procedures

A gravity wall is a type of floodwall. Refer to section 6.1 for guidance on concrete floodwall design, including potential failure modes, methods of analysis, load conditions, and pile foundations if needed.

### 6.3.2 Toe Protection

Wave action under normal conditions and during storm events can cause erosion at the waterside toe of the wall. Toe protection commonly includes a sheetpile cutoff wall along the toe to prevent undermining combined with additional rock revetment armoring. Some factors contributing to scour include wave breaking on the wall at low tides, wave runup and backwash, wave reflection, and the nature and grain size of the material at the toe. In addition to the hydraulic forces at work, the changing configuration of the area fronting the wall over time can contribute to scour. More detailed information and procedures for evaluating and addressing scour can be found in EM 1110-2-1614 (USACE, 1995a).

### 6.3.3 Controlling Runoff and Overtopping Resilience

Wall design should include provisions for erosion protection and drainage of the landside due to potential wave splash and overtopping. Provisions may also be needed for penetrations through the wall as part of managing interior drainage within the leveed area. Penetrations may include gravity drainage pipes with suitable backflow prevention check valves on the waterside, positive close gate valve on the landside, and discharge piping from drainage pump stations.

## 6.4 Demountable Floodwalls

Demountable floodwalls can be employed in situations where a permanent structure is undesirable. Such situations may include:

- Undesirable loss of ability to see beyond the line of the floodwall.
- The need to avoid restricting access and operation of existing facilities.
- The need to maintain current vertical clearances for existing overhead facilities during non-flooding conditions.

Typical demountable floodwalls are shown in Figure 7-30. Note that the demountable floodwall, as described in this section, is designed as a permanent part of a given project. This should not be confused with temporary measures that can be employed for use during floodfight activities.

### Figure 7-30: Example of Demountable Floodwall


*Placement of the 17th Street demountable wall in Washington, D.C.*

Advantages and disadvantages of demountable floodwalls are summarized in Table 7-13.

### Table 7-13: Advantages and Disadvantages of Demountable Barriers

| Advantages | Disadvantages |
|---|---|
| • Generally robust and well-engineered | • Large storage area needed |
| • Good resistance to loading and debris impact | • Heavy transportation and lifting requirements[1] |
| • Durable | • Long installation and mobilization period[2] |
| • Can be increased in height by adding panels up to the height of the frame depending on predicted flood level | • Permanent parts susceptible to damage and vandalism |
| • Very low seepage through the structure | • May not be appropriate in coastal areas subject to significant storm surge and waves |
| | • May not be suitable where vessel or barge impact is possible |

Notes to table:

1. Commercially available devices may require lighter equipment.
2. Commercially available devices may have shorter installation times.

The selection of the type of demountable barrier will depend on many of the same type of factors that should be considered for closure structures (section 7.2). In particular the following should be considered:

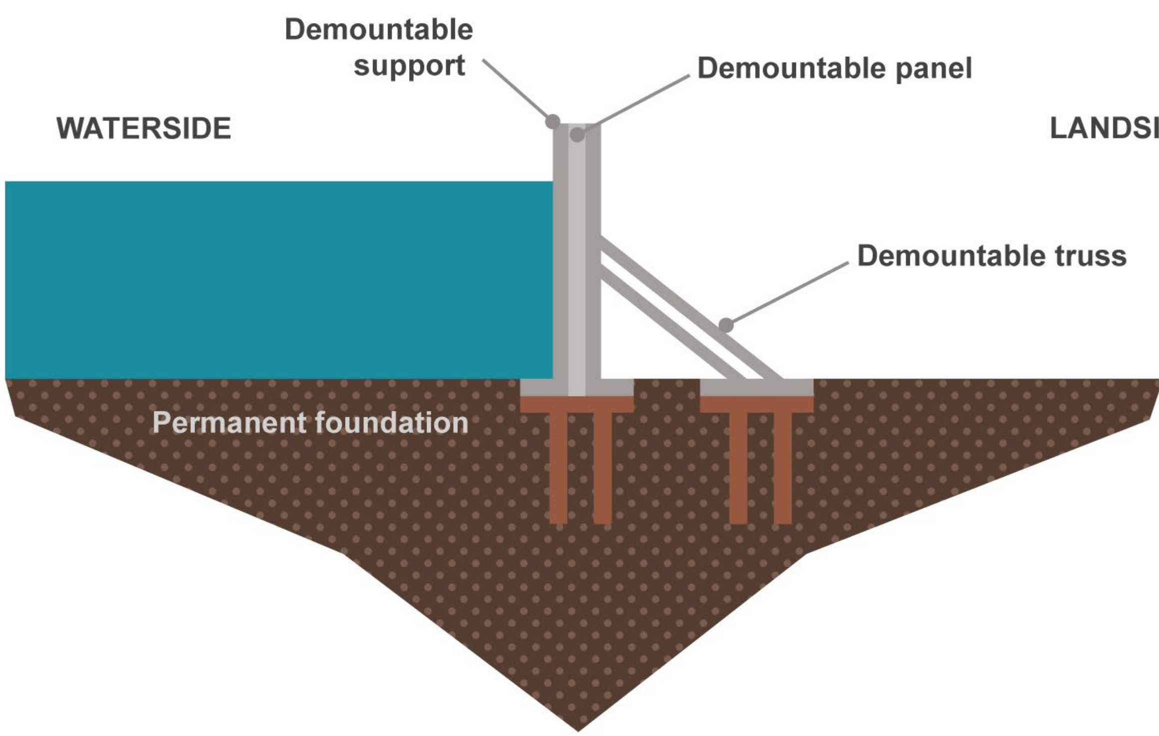
- Time available for installation.
- Storage when not in use.
- Equipment and manpower needs for mobilization and installation.
- Time for which the opening will remain closed prior to, during, and after the flooding.
- Ability to test and conduct emergency exercises.

### 6.4.1 Design

Figure 7-31 is a typical cross section of a demountable barrier showing the various components that may be needed.

A typical installation may include vertical posts (stanchions) placed at intervals that support a single rigid panel or multiple rigid panels that interlock with the posts. The panels are typically metal (aluminum). Seals are required between the panels and the posts, as well as between the panels and foundation to ensure watertightness. Depending on the height of the barrier and the design loading, it may be necessary to provide bracing struts or a truss to support the posts.

**Figure 7-31: Typical Demountable Barrier Cross Section**



A foundation block of concrete is typically constructed for support and to resist sliding and overturning loads. For removable barriers, embedded anchorages can be used so that the posts and bracing can be brought to the site and attached to the foundation when needed. Posts and bracing can also be permanently installed in the foundation if desired.

Special attention should be paid to the closure at the ends of the demountable barrier. The barrier should firmly tie into other risk-reduction features (e.g., embankment levees, floodwalls) or into existing topography via a reinforced concrete abutment wall that prevents end-around seepage. Rock slope protection should be placed at closure locations on the waterside to prevent erosion.

The design process should be iterative, considering various combinations of post spacing and panel component heights to find an economical design with reasonable component weight to facilitate installation and removal.

Overtopping resilience should also be considered, as for other types of floodwalls (section 6.1.2). O&M manuals prepared by the designer should include all facets of the demountable floodwall structures, including parts, diagrams, installation procedures, inspection, and maintenance schedules.

### 6.4.2 Other Considerations

The area selected for storing components should be close to the installation site. It should provide adequate protection of the components from weather damage and vandalism.

The entity responsible for maintaining, installing, and removing the barrier should be clearly identified along with the chain of communication with the agencies responsible for ordering installation of the barrier. The maintaining entity should have proper equipment and personnel qualified in the installation process. A test installation of a short section of barrier should be performed annually. For removable barriers, the foundation attachments and foundation for the barrier should be inspected annually prior to the flood season.

# 7 Closure Structures

Closure structures (**Chapter 2**) are used to close gaps in the levee alignment, such as where infrastructure (e.g., a road or railroad) or another water body (natural or human-made) crosses or intersects the alignment. This is done to prevent water from entering the leveed area during high water. Closure structures also may be used to provide access through a levee, such as for maintenance or recreation. The preference is to maintain levee continuity, for example, by routing a road over the levee alignment. However, when this is not possible, a closure feature provides an alternative. Figure 7-32 shows examples of closure structures.

Deciding when a closure structure is needed and where to place it along the levee should be carefully considered to ensure the best possible design for the full lifecycle of a project. Design of a steel closure structure should adopt either allowable working stress or load and resistance factor design depending on which approach best suits the project-specific requirements.

O&M manuals should include all facets of the selected closure structures, including parts, diagrams, lubrication points, inspection, and maintenance schedules.

**Figure 7-32: Example Closure Types**



*Examples of two different types of closures. A swing gate closure across a roadway along the Ohio River in Louisville, Kentucky, and a sector gate closure across a waterway in New Orleans, Louisiana.*

## 7.1 Selection of Type of Closure Structure

As described in **Chapter 2**, there are three main categories of closure structures:

- Movable gates (roller, swing, trolley, vertical lift, sector, miter).
- Structural assembled closures (stoplogs of timber, metal, or concrete).
- Earthen assembled closures (using sandbags, soil/gravel baskets, or earthen fill with plastic covering).

The type, location, and number of closures used in a project are important decisions to ensure the levee system will be able to perform as intended during a flood event. Factors to be taken into account in the selection of closure type should include:

- **Time available for installation**.
  - Rapid closure requirements. Levee systems that provide risk reduction against flashy water sources (those that rise and fall within a matter of hours) should only utilize movable gates that can quickly be closed, as there will be insufficient warning time to install other closures. Structural or earthen assembled closures may be used if an extended warning time is available.
    - When a temporary closure is needed across frequently used roads or railroad tracks, movable gates may be the best option, as they can be closed quickly, allowing the greatest amount of time for traffic to evacuate out of an area that could be inundated. This is an important decision factor for active railroad lines.
  - Limitations on manpower and equipment. Where systems have multiple closures as part of a single levee system and there is limited manpower and equipment given the available time to close, the number of closures included in the design may need to be restricted.
- **Time for which the opening will remain closed prior to, during, and after the flooding**. Where closure of an opening is feasible for a longer period of time, structural or assembled closures may be acceptable and cost-effective. Otherwise, use of movable gates are recommended.
- **Size of opening**. Structural and earthen assembled closures may be used across a wide range of opening widths and heights. When selecting movable gates, width and height will affect the selection of the type. For example, mechanical swing gates, though widely used, are more limited in the length of span they can close as compared to rolling gates, which are available in a wide variety of lengths and heights.
- **Storage when not in use**. Movable gates need adequate real estate for storage when not in use.
- **Equipment and manpower needs for mobilization/installation**. Movable gates that require large equipment, such as cranes, for installation should be avoided when possible. These can be designed where real estate is available to store the gate while not in operation.
- **Capital, operation, and maintenance costs**. For some types of movable gates, the O&M costs may be significant and can result in higher expenses over the levee system's service life. Repair of deteriorated concrete and embedded steel components, installation practice and training, availability of future replacement parts, storage, and manpower requirements are all costs that should be considered during the design phase of a project. This is particularly important when considering the use of proprietary closure systems where the availability of replacement parts and supplier support cannot be guaranteed into the future.
- **Ability to test and conduct emergency exercises**. Closures for transportation corridors often have limitations or constraints to being able to test closures. These often require extensive coordination in order to be able to temporarily close the corridor, especially in a non-emergency situation.

These factors have been taken into account in the preparation of Table 7-14, which may be used as an initial guide to assist with selecting closure types. Ideally, the final decision should be supported by project-specific operational equipment, time, and manpower analysis completed to support an evaluation or design risk assessment.

**Table 7-14: Recommended Closure Types for Different Design Scenarios**

| Hydraulic Hazard Condition | Closure Location or Height | Recommended Type of Closure Structure[5] |
|---|---|---|
| "Flashy" stream or river | Roadway[1] or railroad[2] | Movable gate (swing, roller, or trolley) |
| "Slow rising" stream or river | Closure height > 4 feet[3] | Movable gate (swing, roller, or trolley) or structural assembled closure[4] |
| | Closure height < 4 feet | Movable gate (swing, roller, or trolley), structural assembled closure, or earthen assembled closure (likely soil/gravel baskets, earthen fill with plastic, or sandbags) |
| Coastal storm risk management systems[6] | Navigation channel with reverse loading | Sector type of movable gate |
| | Navigation channel only loaded from one side | Sector or vertical lift type of movable gate |
| | Structures on land | Movable gate (swing, roller, or trolley) |

Notes to table:

1. Careful consideration is required when deciding to design a closure versus raising the roadway grade. Raising the grade can eliminate a closure. This may be a critical factor for communities where a roadway is an important evacuation route.
2. Raising railroad grade could theoretically eliminate a closure, but the change of grade would have to be carried over such great distances that railroads generally reject this design suggestion.
3. Applies to both roadways and railroads.
4. Installation of structural assembled closures takes a much larger contingent of manpower to install compared to swing gates or rolling gates. A decision to use this type of closure should be based on an understanding of local manpower resources available during a flood emergency.
5. Selection of the type of closure may be affected by its length since greater local manpower resources may be required to implement some types of longer closures.
6. Rapid intensification of hurricanes points to limiting closure types to those that can be installed very quickly using the least manpower resources.

## 7.2 Closures Across Transportation Corridors

Operating and emergency planning should include coordination with the applicable transportation agencies to ensure operations do not interfere with the closures. This should also include coordination procedures for periodic testing of closures, including emergency response exercises.

When possible, the levee alignment should be perpendicular to the roadway or railroad where any closure is required. This minimizes the width of the closure opening, which can lower construction costs and shorten installation time of the closure. Benefits of this approach include reducing flood risk and operational interference of the railroad/roadway.

### 7.2.1 Vehicular Closures

Coordination with representatives from the local community should occur in order to evaluate the community's needs and set the priorities for which streets that cross the levee alignment require closures, may need to be rerouted, or may need to be permanently closed. These decisions also may impact future land use and emergency evacuations.

Generally, gap closures crossing roadways should be perpendicular and at grade. If the roadway is designated as an evacuation route or emergency access, the closure type may well be a swing, roller, or trolley movable gate. Opening widths need to accommodate removable vehicle crash tested barriers, per the governing highway authority requirements. Thus, swing gates may not be ideal, as a large section of removable vehicle crash barriers will be required. Other items to consider are the highway speeds, drainage, and grading.

The opening widths for vehicular gates should comply with the American Association of State Highway and Transportation Officials requirements, as well as with state and local regulations. For overhead roller gates, the minimum vertical clearance between the crown of roadways and fixed overhead components of closures should not be less than 14 feet. Clearances should be coordinated with, and approved by, the relevant transportation organization. Warning signs are required at overhead roller gates.

### 7.2.2 Railroad Closures

Minimum horizontal and vertical clearances should not be less than that required by the American Railway Engineering and Maintenance-of-Way Association. The normal minimum width of opening provided for railroads is approximately 20 feet for each set of tracks involved in the closure.

The railroad authority should be involved as early as possible in the formulation and design process since the American Railway Engineering and Maintenance-of-Way Association's general guidelines may not be sufficient to satisfy site-specific railroad operations. During design reviews, providing construction phasing plans that highlight sequencing and track down time may be necessary. Other common requirements consist of:

- Crossing perpendicular to the tracks.
- Locating the tops of foundations below the ballast and sub-ballast material.
- Limiting the width of the sill so it fits between the clear distance between railroad ties.
- Taking account of robotic train operations and gas lines used for switch heaters.

### 7.2.3 Waterway Closures

Waterway closures prevent water from flowing into waterways that intersect the levee alignment. The operability and closure time are key factors when selecting a gate type. Impacts on drainage and navigation will dictate the operation. Common types of movable closures for waterways include:

- **Sector gates**. These gates are the most versatile. They can be designed to operate in adverse conditions and do not impose height restrictions.
- **Vertical lift gates**. These gates can be operated by fixed overhead cranes or by moving gantry cranes. The disadvantages of the vertical lift gates are the machinery requirements and the overhead restriction.
- **Buoyant gates**. These gates also are used but require a steady-state condition to operate, should be closed far in advance of high water, and cannot be opened until the opposing water stages drop.

### 7.2.4 Pedestrian Closures

Closure structures for pedestrian access ways through floodwalls can be simple gates, typically of steel, with appropriate seals to ensure water tightness. Often these are closeable manually. The form of the gate should be agreed with the local community, including the proposed approach to O&M.

For earthen embankments, closures are rarely provided, although ramps may be required to facilitate people walking up and over the embankment.

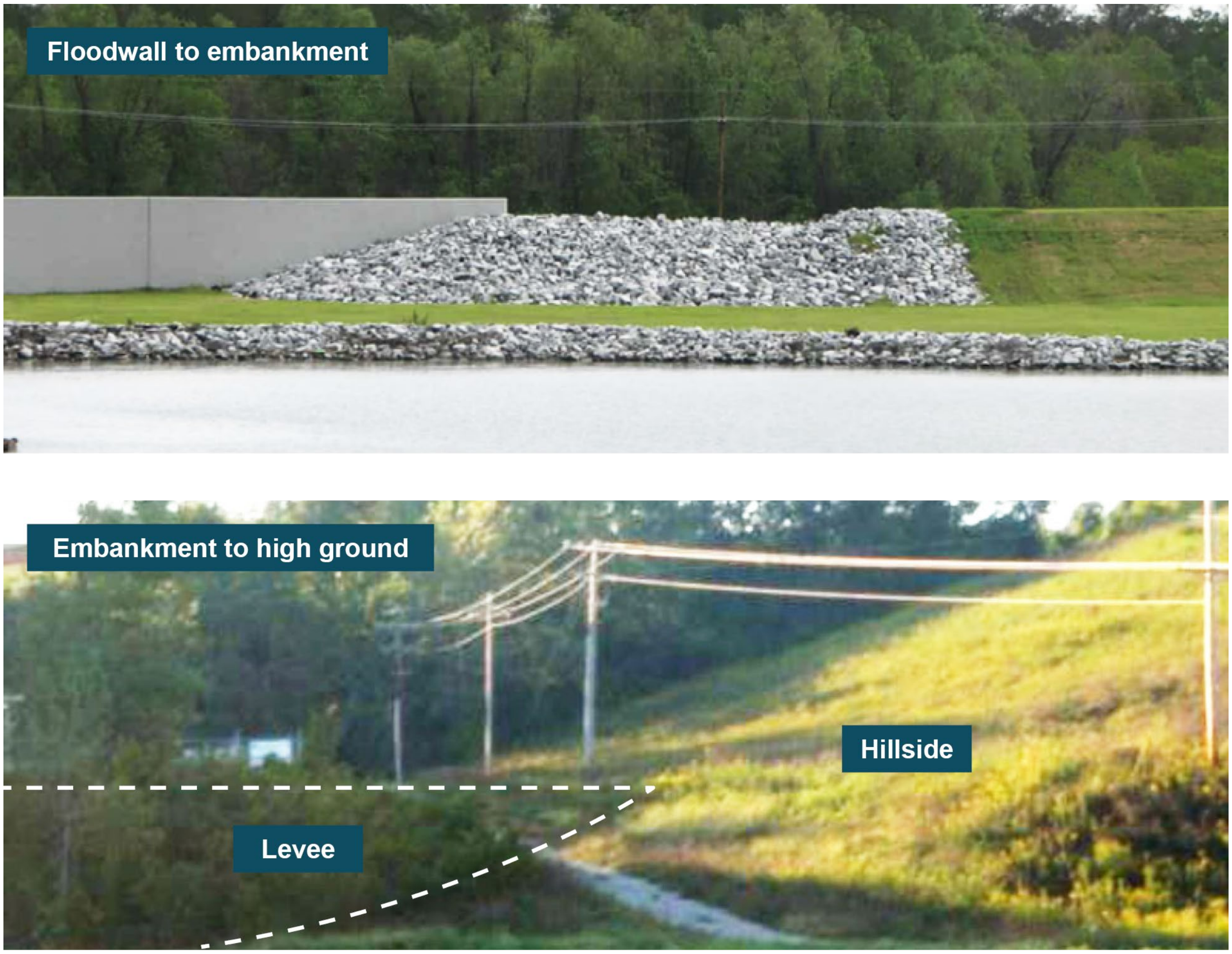
# 8 Transitions

Loss of integrity at the **transition** locations between different features along the levee alignment can lead to failure of the levee at these points. This risk factor should be addressed in design. Transition locations may include:

- Earthen embankment to floodwall (concrete or steel) transitions.
- Earthen embankment or floodwall transition to concrete closure structures.
- Earthen embankment or floodwall tying into existing natural grade.
- Earthen embankment or floodwall tying to other existing infrastructure, such as bridge abutment walls or road embankment fills.
- Encroachments by pipe and culvert systems into earthen embankments.

Figure 7-33 shows two examples of transition locations.

**Figure 7-33: Examples of Transition Locations**




*Two examples of floodwall transition locations. A floodwall transition to an embankment with riprap providing erosion protection and a floodwall transition to embankment into high ground.*

## 8.1 Design Principles for Transitions

- **Avoid or minimize transitions when possible**. When carrying out levee rehabilitation, it may be possible to remove existing transitions rather than trying to control or address their impacts. Alternatively, a transition may be relocated to reduce loading or improve resilience.
- **Ensure the transition is not the weakest link** in the levee chain, and that this remains the case through the whole life of the levee, taking account of deterioration processes. This may involve being more conservative in the design of the transition than for the adjoining levee segments.
- **Transitions should be gradual**, both in terms of external geometry and also materials and structure types. Abrupt changes in direction increase the turbulence of water flow and lead to increased risk of external erosion. This risk can be minimized by providing sufficient overlap of structures and surface protection systems and avoiding abrupt

changes. In this regard, transitions should be considered in three dimensions (not just in plan or in cross section).

- **Loading conditions**. A range of loading scenarios should be considered including normal operating conditions, design flood events, and extreme events that exceed the design event. The following hydraulic parameters need to be considered:
  - Velocity and direction of flow, including turbulence and possible sediment transport.
  - Water level changes, including any waves and their characteristics.
  - Hydraulic head along and across the transition and the resulting potential for hydraulic separation and uplift.
- **All potential failure mechanisms** should be addressed including:
  - External erosion, which increases at transitions where there is increased turbulence.
  - Internal erosion, especially at cracks at interfaces between earthen structures and hard structures due to increased rates of seepage.
  - Differential settlement.
- **Consider deterioration processes** that could compromise performance over time. For example:
  - Seasonal shrinkage/swelling of clay soils leading to desiccation cracking.
  - Seepage and/or hydraulic separation.
  - Local settlement leading to the possibility of localized overtopping flow.
- **Levee modifications or improvements** may:
  - Cause short-term performance reductions at disturbances to the existing levee system, for example until grass has re-established or the consolidation of the ground has completed.
  - Introduce new transitions (including at existing transitions), with related impacts on levee performance.

## 8.2 Dealing with Specific Mechanisms

When joining a levee embankment with a concrete or sheetpile structure or floodwall, concerns that should be considered in design of the junction include embankment slope erosion, seepage and internal erosion and differential settlement.

### 8.2.1 Embankment Slope Erosion

Turbulence may result at the junction between earthen embankments and hard concrete or steel structures (floodwalls, closure structures, etc.) because of changes in the geometry between the levee and the structure. This turbulence causes scouring of the levee embankment if slope protection is not provided. Rock, concrete, or proprietary erosion control systems for slope protection should be considered for the levee embankment at such locations.

### 8.2.2 Cracking and Hydraulic Separation Leading to Seepage and Internal Erosion

Cracking and/or hydraulic separation at the interface between the embankment fill and the hard structure (e.g., floodwall or drainage control structure) can increase the risk of seepage leading to internal erosion in the form of concentrated leak erosion.

Seepage analyses should be performed to establish the required minimum embedment of a floodwall into an earthen embankment, to reduce seepage pressures and the potential for internal erosion. Seepage analyses also should be performed where earthen embankments or floodwalls terminate into existing topography, to determine whether treatment is required to control or prevent end-around seepage in the hillside.

Concrete floodwalls, wingwalls, and sheetpiles may be extended beyond the concrete structure well into the earthen structure (e.g., along the levee centerline) to increase the length of seepage paths and reduce seepage gradients which might induce internal erosion.

Thorough compaction of the levee embankment at the junction of the concrete structure to ensure firm fill contact with the structure and levee is essential. This helps to decrease the hydraulic conductivity of the embankment material and reduces the risk of cracking or hydraulic separation. In this situation, the exterior of the abutting end walls of the concrete structure should be battered at an angle of 10 vertical to 1 horizontal to assist in ensuring adequate compaction and a firm contact between the structure and the fill. Compaction equipment should be selected based on available working room. Heavy compactors should be used wherever possible, except near concrete structures where light equipment or hand tampers should be used to avoid locking in high residual stresses in the structure. See EM 1110-2-1911 (USACE, 1995) for further details.

### 8.2.3 Differential Settlement

Differential settlement can result from unequal consolidation of soft foundation soil at the transition between a relatively heavy levee embankment and a relatively light concrete floodwall or closure structure. Such differential settlement can locally increase the rate of overtopping during a flood event and encourage failure by external erosion of the rear face of the embankment.

Thorough compaction of the embankment material is important at locations of potential differential settlement. Furthermore, given that hard structures such as floodwalls with landside scour protection may be more resilient to overtopping than embankments, it may be desirable to add an overbuild settlement allowance to the embankment to ensure that overtopping takes place preferentially over the more robust hard structure. Further guidance on transitioning procedures for a junction between a levee embankment and a floodwall is available in EM 1110-2-2502 (USACE, 1989a).

# 9 Seepage Control Features

Seepage control features reduce the probability of levee breach arising from internal erosion or foundation erosion from throughseepage or underseepage. A seepage cutoff wall significantly reduces or eliminates the seepage, whereas other options, such as seepage berms, only control the seepage.

- **New embankment levees** to be constructed using low permeable fill meeting the requirements discussed in section 5.1.3 should not be susceptible to internal erosion by throughseepage. The embankment may be homogeneous or zoned. However, a risk of foundation erosion still may exist because of uncontrolled underseepage.
- **Existing levee embankments** may be at risk of potential failure because of internal or foundation erosion potential failure modes, or both, as indicated by historic poor seepage performance or by post-construction geotechnical evaluations. A seepage cutoff wall may be a risk reduction option.
- **Floodwall** seepage control features can be introduced to deal with similar potential failure modes as for embankments. Cutoff walls should be tied into the floodwall to prevent seepage through the interface between the cutoff and the above-grade portion of the wall. Note that the seepage berms discussed in section 9.3 generally are not used with floodwalls because of right-of-way restrictions. For detailed design of seepage control features, reference should be made to the seepage control criteria in EM 1110-2-2502 (USACE, 1989a) and EM 1110-2-2100 (USACE, 2005).

## 9.1 Seepage Analysis

Two-dimensional, finite element analysis software that analyzes groundwater seepage and excess pore pressure dissipation conditions in porous materials such as soil and rock should be used for analysis of seepage. The accuracy of the analyses depends on the extent and quality of subsurface data and material testing available to make the seepage models. Guidance on performing seepage analyses for levees is provided in EM 1110-2-1901 (USACE, 1986). Experienced professionals should oversee interpreting subsurface data, defining soil stratification, and assigning seepage properties to the soil strata.

### 9.1.1 Potential Failure Modes

Levee collapse and breach because of seepage results from:

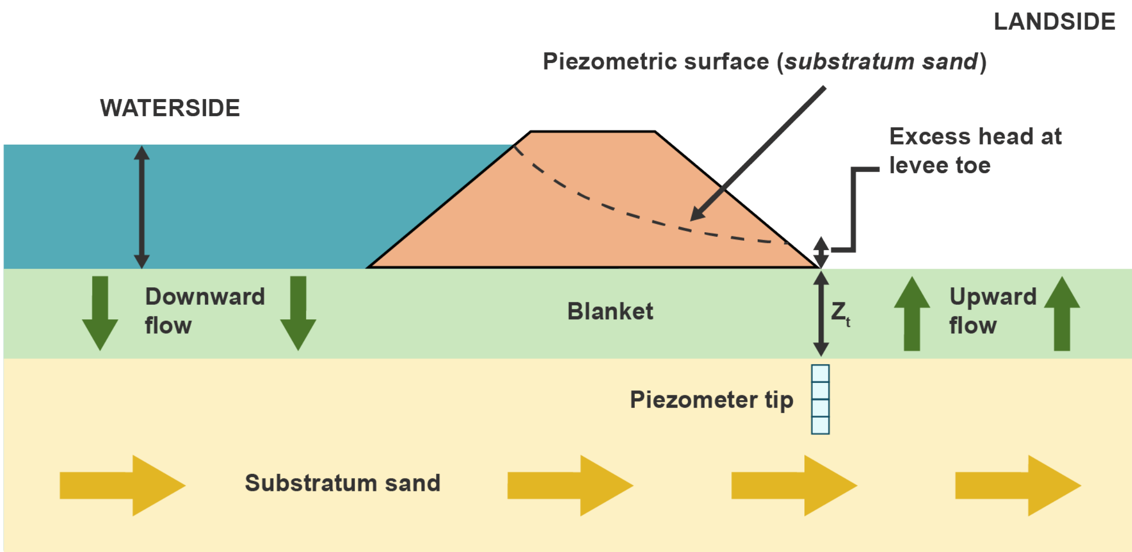
- Backward erosion piping under the levee.
- Throughseepage reducing the soil strength of a levee embankment, causing sloughing and erosion of the landside slope.
- Concentrated leak erosion through pre-existing cracks/flaws in the levee.

Problematic geologic conditions for backward erosion piping include blanket conditions where layers of lower permeability material overlie more permeable deposits, permeable embankment materials, and daylighting permeable layers.

Problematic conditions for concentrated leak erosion relate to the presence of flaws such as animal burrows, tree roots, pipe encroachments, cracking, and/or hydraulic separation at interfaces between earthen embankments and hard structures. Approaches to dealing with concentrated leak erosion are discussed in section 8 on transitions.

**Blanket conditions**. Figure 7-34 shows a levee section where the water is on the left side and the leveed area is on the right. The green layer is a relatively impermeable clay layer, and the yellow layer is a more permeable sand layer. During high water, flow occurs in the sand layer and water pressure will push upwards on the clay layer, or blanket, on the landside. If the pressure is high enough, the water breaks through the clay layer and carries sand with it, creating a boil and potentially undermining the levee as the piping progresses backwards.

### Figure 7-34: Blanket Conditions




Permeable levees—consisting of sand, gravel, non-plastic silt, or continuous layers of these materials—may pose throughseepage hazards. These result from flow through the levee, potentially carrying material and creating backwards internal erosion, causing collapse of the levee. Another potential failure mode for permeable levees is degradation of the landside face caused by throughseepage daylighting on the face, causing erosion. This is a progressive failure mode where the landside slope becomes unstable because of progressive steepening, caused by the degradation.

Daylighting permeable layers, sometimes referred to as leaking layers, are permeable layers directly below the levee that daylight and are exposed at the ground surface on both sides of the levee. These layers allow direct flow under the levee from the waterside to the landside. If sufficient flow and water pressure are present, these conditions can move foundation material toward the landside, creating backwards internal erosion underlining the levee.

### 9.1.2 Analyses Approach

The seepage analysis objectives are to:

- Estimate steady-state phreatic levels and pore pressures in levee embankment and foundation soils for selected water level conditions; the resulting pore pressure information is also used for slope stability analyses.
- Estimate the phreatic surface breakout location.
- Where a blanket layer is present, calculate the average vertical hydraulic exit gradients.
- Compare the resulting hydraulic exit gradients with design criteria.

Regulating agency guidelines typically require a steady-state seepage analysis, even if flood events are short duration. Analysis sections should be selected taking account of reaches that are significantly three-dimensional (e.g., where there are bends and meanders in the levee).

**Material properties selection**. Steady-state seepage analyses typically require input of the following material properties:

- Horizontal hydraulic conductivity under fully saturated conditions (kh).
- Ratio between vertical and horizontal conductivities (anisotropic ratio) (kv/kh).

The designers may select material properties to be used for analyses using a variety of methods including:

- In situ hydraulic conductivity tests including a pumping test.
- Laboratory hydraulic conductivity tests.
- For granular soils, empirical methods such as the Kozeny-Carman equation, in combination with the results of gradation tests.
- Empirical charts that relate hydraulic conductivity to void ratio and the effective grain size, d10.

Guidance on selection of material properties is available in the Guidance Document for Geotechnical Analysis (California DWR, 2015).

**Three-dimensional effects**. Modeled two-dimensional seepage gradients should be increased where appropriate to take account of three-dimensional effects. State of California Department of Water Resources (California DWR, 2015) and (Jafari *et al.*, 2015) provide some rules of thumb for adjustments.

### 9.1.3 Design Criteria

Levee underseepage evaluation is based on the critical vertical hydraulic gradient/vertical effective stress factor of safety method (Terzaghi, Peck and Mesri, 1996). Originally, levee underseepage criteria were developed for a horizontal ground surface, with a vertical hydraulic gradient assessed from head loss across a blanket at the levee toe. This head loss is divided by the average landside blanket thickness (noted as "$Z_t$" in Figure 7-34), yielding a hydraulic gradient often referred to as the 'exit gradient.' See EM 1110-2-1913 (USACE, 2000) for guidance on allowable seepage gradients. The exit gradient is then compared with the critical hydraulic gradient for the situation. Critical gradients may need to be adjusted in three-dimensional situations, as discussed for example in (Van Beek *et al.*, 2015).

Throughseepage evaluation is not required for a new levee if it is constructed using low-permeable fill meeting applicable criteria. For an existing levee, if a phreatic surface daylights on the landside slope of a levee under a steady-state seepage condition, it may indicate a potential for throughseepage distress. Low-plasticity, or erodible soils (e.g., silt and sand) are more susceptible to piping and surface erosion than plastic soils (e.g., clays, clayey sands, clayey gravels), and thus the designers should identify this breakout condition for erodible soils as a throughseepage deficiency. Designers should consider the available historical construction data to identify whether zones of potential erodible material are encapsulated by non-erodible soils in the exterior of the levee embankment.

## 9.2 Seepage Cutoff Walls

Seepage cutoff walls significantly reduce or eliminate embankment throughseepage and foundation underseepage, addressing the risk of failure by backward erosion piping. Design of seepage cutoff walls requires a well-informed understanding of geologic foundation conditions to evaluate the required wall depth and composition. This understanding dictates the appropriate design and construction methods. Table 7-15 summarizes the seepage cutoff wall design elements, advantages, and disadvantages.

**Table 7-15: Seepage Cutoff Wall Design Elements**

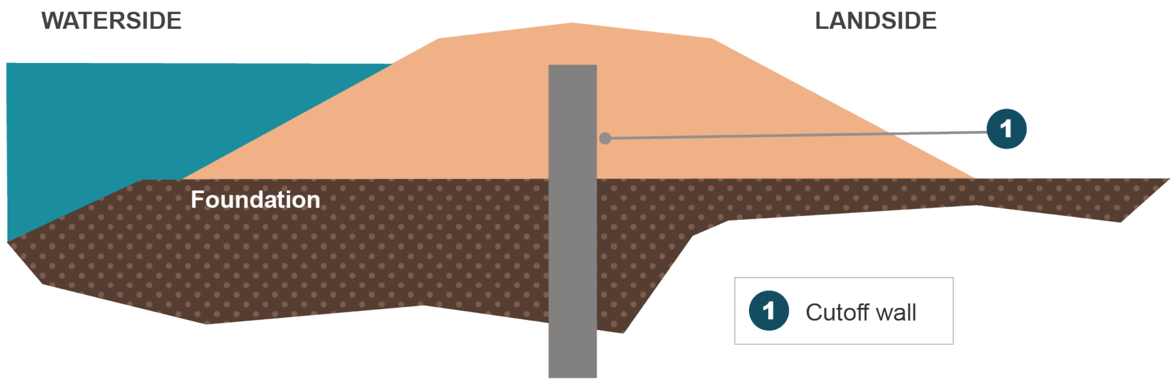
| Seepage Control Feature | Associated Potential Failure Mode | Design Elements | Advantages | Disadvantages |
|---|---|---|---|---|
| Cutoff wall | • Internal erosion piping<br>• Foundation erosion piping<br>• Slope stability | • Alignment<br>• Depth<br>• Composition<br>• Construction method | • Cuts off seepage<br>• No maintenance<br>• No additional right of way | • Cost<br>• Construction risk<br>• Higher groundwater levels during times of low flow |

### 9.2.1 Alignment and Other Design Considerations

For a new levee, Figure 7-35 shows the seepage cutoff wall constructed on the centerline in the inspection trench area. The inspection trench should be backfilled with compacted embankment fill to foundation grade before the slurry cutoff wall is installed. The inspection trench and cutoff wall can be moved laterally from the centerline towards the waterside if necessary. The inspection trench also helps with eventual trench stability and to prevent caving during construction.

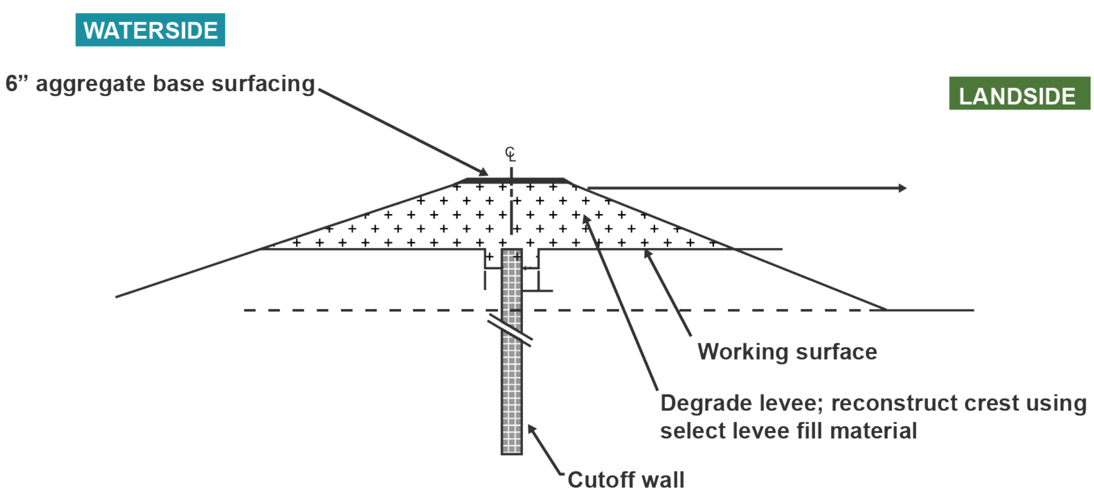
An open trench stability analysis should be completed especially for any cutoff wall constructed at the toe of the levee which has high shear forces from the sloping embankment. For analytical methods for trench stability or seepage/stability modeling, reference should be made to EM 1110-2-1901 (USACE, 1986).

#### Figure 7-35: Typical Seepage Cutoff Wall—New Embankment




For an existing levee, Figure 7-36 shows an example seepage cutoff wall constructed on the centerline of the levee. In addition to remediating underseepage in the levee foundation, the cutoff wall also remediates throughseepage because it extends upward, through the embankment to the working platform. The top of the cutoff wall should tie into the compacted embankment fill to prevent seepage over the top of cutoff wall.

#### Figure 7-36: Typical Seepage Cutoff Wall—Existing Levee




During construction of cutoff walls in existing levees, degrading the existing levee crown often is required to establish a working surface wide enough for the cutoff wall construction equipment, with some additional space on at least one side for construction traffic to pass. The typical degrade should be at least one-third of the height of the levee to limit fracture/break-outs—the actual degrade depends on crown width, trenching equipment used, and levee side slopes. The degraded top of the levee should be reconstructed using approved embankment fill. Figure 7-36 shows one method of installing the cutoff wall in a trench excavated below the levee working surface to prevent seepage over the top of the cutoff wall. The trench should be backfilled with embankment fill before wall construction.

Another option for an existing levee requiring underseepage rehabilitation only is to install a waterside toe cutoff wall. This can be an attractive option for existing floodwalls. Toe walls also eliminate the need to degrade the existing embankment levees but require good access to the waterside toe of the levee for construction. This can be a viable option compared to levee centerline walls as long as the waterside slope of any embankment levee is low permeable fill, and the top of the toe wall terminates in that fill.

### 9.2.2 Depth

The depth of the initial inspection trench should be sufficient to encapsulate the cutoff wall and any expected consolidation/settlement of the wall during initial set.

The bottom of the cutoff wall for a new or existing levee should be set at an elevation that prevents seepage flow under the bottom of the wall. Cutoff wall depths should be determined based on the foundation stratigraphy and thus can vary along the length of the wall. The intent of the wall is to cut off seepage flow in relatively permeable layers (aquifers) underlying the embankment. The bottom of the wall should penetrate into a thick layer (not seam) of low permeable soil (aquiclude) beneath the aquifer layers. A typical minimum penetration into the permeable layer of materials should be 5 to 10 feet. To achieve seepage cut-off, the depth of embedment into the aquiclude may need to vary along the length of the wall as the nature of the aquiclude material varies. Geotechnical drilling and sampling along the wall alignment should be analyzed in design to estimate the required wall tip elevation profile shown in the drawings.

Practical limits exist to the depth of cutoff walls. Construction technology continues to develop, but generally walls deeper than 140 feet are not practical. Walls deeper than 70 feet may require more expensive construction methods.

### 9.2.3 Composition

Cutoff walls for a new or existing levee may consist of a variety of materials. These include structural elements, such as steel sheetpiles, concrete walls, or mixed in-place walls using slurry trench methods. Mixed-in-place walls can use different mixes, depending on existing in situ soils and the design requirements for wall permeabilities and strengths. Typical mixes for slurry walls include soil-cement, soil-cement-bentonite, and soil-slag cement-cement-bentonite. Further information on slurry walls is available in Slurry Walls: Design, Construction, and Quality Control (Paul, Davidson and Cavalli, 1992).

The type of wall and composition is dictated by the required depth of the wall, constructability, and the required wall permeability and strength properties. For open trench wall configurations, a stability analysis should be conducted as part of the design to evaluate the stability of the open trench. Steel sheetpiles and concrete cutoff walls may be constructable only to limited depth, depending on geologic conditions (e.g., see detailed information in (Bruce, 2013)).

### 9.2.4 Advantages and Disadvantages

The main advantages of seepage cutoffs are if they are designed and constructed properly, they virtually eliminate seepage through or beneath the levee. After being constructed, the walls require no maintenance. Cutoff walls also require no additional right of way because they are constructed within the planned or existing embankment footprint. Slurry cutoff walls also allow confirmation of geologic layers by observing excavated material as the trench is excavated.

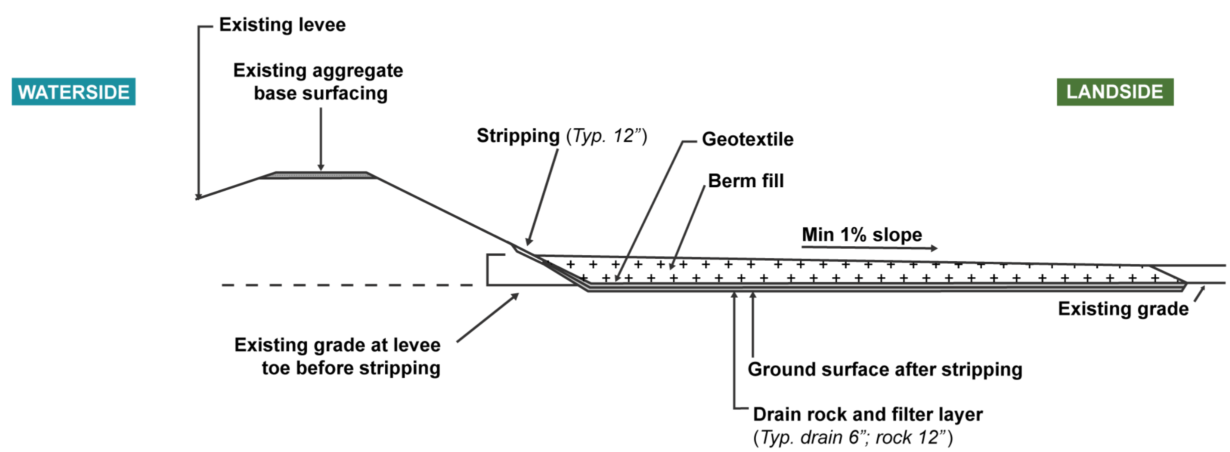
Disadvantages include the following:

- Cutoff walls interrupt regional groundwater flow during periods of low water and thus lead to higher groundwater levels.
- Historically, landside seepage berms have been cheaper than seepage cutoff walls. However, in some areas of the U.S., costs now are comparable because new construction methods have been developed. Also, in some areas, steel sheetpile cutoff walls are competitive with slurry cutoff walls.
- Need for high quality construction including construction quality control.
- Sheetpiles generally are driven or pushed into the ground, and no method is available to confirm the geologic layers penetrated.
- Concrete cutoffs are limited in depth because of trench instability.
- Slurry cutoff walls:
  - Require good control of materials.
  - Some geologic conditions may make slurry cutoff walls infeasible.

## 9.3 Seepage Berm

A seepage berm is intended to mitigate the risk of embankment breach because of backwards erosion (piping) of the foundation soils from underseepage. Figure 7-37 shows a seepage berm installed on the landside of a levee. Such a berm is required when the seepage gradient at the landside toe of the levee exceeds applicable criteria.

**Figure 7-37: Typical Seepage Berm**




A seepage berm places weight on top of the landside ground to reduce the potential for heave and thus the migration of underlying soil to the ground surface in the form of sand boils. Depending on the geologic conditions, the required width of the berm may become impractical.

Upward movement of foundation soils to the ground surface still may develop at or outside the landside toe of the berm.

For a new embankment levee, a seepage berm may be an option in cases where a seepage cutoff wall technically is not feasible or is cost prohibitive. Table 7-16 summarizes seepage berm elements and advantages and disadvantages. Design elements are discussed in the following paragraphs.

**Table 7-16: Seepage Berm Feature Design Requirements**

| Seepage Control Feature | Associated Potential Failure Modes | Design Elements | Advantages | Disadvantages |
|---|---|---|---|---|
| Seepage berm | • Foundation piping breach formation<br>• Embankment piping breach formation | • Drain layer<br>• Width<br>• Height<br>• Composition | • Cost<br>• Lower construction risk | • May not reduce seepage<br>• May still allow boils and require floodfighting<br>• Additional right-of-way required<br>• Potential for maintenance and erosion |

## 9.3.1 Drainage Layers

Seepage berms commonly are drained to facilitate the flow of seepage water away from the levee. If drained, the seepage berm shown in Figure 7-37 may also extend up the landside levee slope (known as a chimney drain extension) to collect any throughseepage. This can be accomplished by extending the drainage layer further up the embankment slope and covering it with a minimum of 2 feet of soil for protection. The chimney section should extend up the levee slope to a minimum of one-third the height of the levee. It can be extended further up the slope, based on performance data or seepage analyses.

The drain usually consists of 6 to 12 inches of highly permeable rock, underlain by a filter layer. Filter compatibility should be verified between the drain rock and filter layer, and between the filter layer and subgrade. Regulating agencies typically do not allow the use of geotextile fabrics on the subgrade as a replacement for the filter layer.

## 9.3.2 Width

The berm width can be established by seepage analyses. Iterative calculations should be used to establish the point at which the gradients at the toe of the berm reduce sufficiently to discourage development of boils. A wider berm also reduces the likelihood of piping to progress under the levee before water levels drop and provides time for floodfighting. A minimum width of 150 feet is common.

### 9.3.3 Height

The height of the berm should be sufficient to prevent heave and reduce seepage gradients to meet criteria. A minimum height of 5 feet at the levee toe is common. A minimum height of 2 feet at the berm toe is also common in order to delineate the limit of the berm for maintenance purposes.

### 9.3.4 Composition

Seepage berms are not structural features and can be constructed of variable materials. However, maintenance requirements should be considered and problematic soils, such as highly plastic clays or organics, should be avoided. The presence of locally available borrow materials should be considered in design; but it is preferable for the seepage berm material to be more pervious than the underlying material or pressures will not be able to dissipate. If impervious soils are used, this will result in higher seepage gradients beneath the berm, requiring significantly longer berms than berms constructed with free-draining soils.

### 9.3.5 Advantages and Disadvantages

The advantages of berms are that:

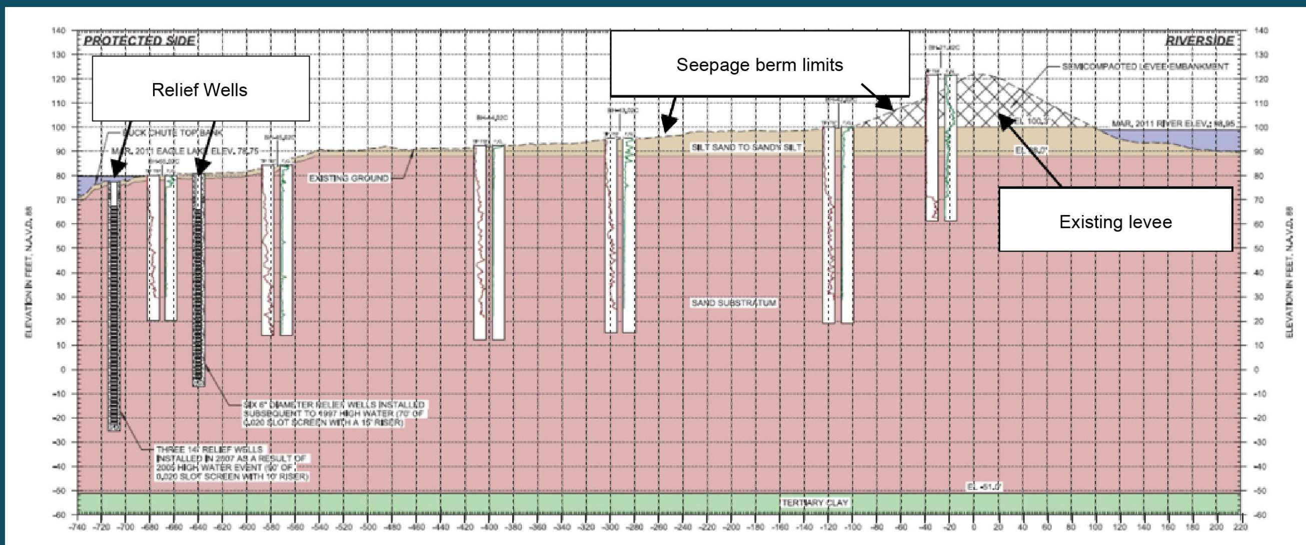
- Any boils which may arise develop further from the levee, reducing the likelihood of backward erosion undermining the levee and allowing time for floodfighting.
- Simplicity of construction being above-grade, results in simpler quality control and higher construction confidence.
- Lower cost. Historically, berms have been cheaper to construct than seepage cutoffs, although the difference in cost has been reduced with recent technological advances for cutoff walls and the challenges of obtaining suitable borrow material for berms.

The disadvantages of berms include:

- Larger footprint, requiring additional right of way. This can be an issue in agricultural and orchard areas because planting can be restricted on the berm.
- Need for maintenance and inspection.
- Difficulties in placing drainage layers for drained berms.
- Water which seeps under the berm to the landside may require control and management.

## MISSISSIPPI LEVEE SEEPAGE BERM

An existing levee along the Mississippi River was re-evaluated and redesigned for seepage issues. The initial plan was to install a seepage berm along the landside of the levee. Existing relief wells were located about 400 feet from the toe of the levee and had historical issues with the well screens clogging. The seepage berm was not planned to extend to the relief wells shown in the cross section. The initial design did not require work in the relief well field to meet the required deterministic seepage design factor of safety.




During the design phase, a flood with a 5-year return period occurred. During monitoring of the flood event, sand boils were observed adjacent to existing relief wells.

After the flood, a risk assessment was conducted and identified a zone of continuous fine sand that extended from the river to the relief wells. This fine sand contained low coefficients of uniformity and was highly susceptible to internal erosion. The conclusions of the risk assessment were that the proposed seepage berm did not reduce the probability of failure in this area and that the probability of failure for an internal erosion failure starting in the relief well field was not tolerable. The design was modified to include a filter blanket that would extend laterally beyond the limits of the fine sand in the vicinity of the relief wells. This filter blanket resulted in a $1.5 million cost increase, but also reduced the risk of failure by 3 orders of magnitude. This is an example of an ‘upscaled’ feature added that would not have been considered necessary just to meet deterministic factor of safety criteria.

## 9.4 Landside Pressure Relief Systems

Landside pressure relief systems can reduce the risk of a breach by reducing seepage pressures at the landside embankment toe while retaining foundation soils. This reduction of seepage pressures at the landside embankment toe reduces the risk of internal erosion. Different pressure relief systems can be considered. The most common types include blanket drains and toe drains to collect throughseepage, and trench drains and relief wells to collect underseepage, which are illustrated in Figure 7-38 and Figure 7-39 respectively.

Table 7-17 summarizes landside pressure relief system elements, and their advantages and disadvantages. Design elements are discussed in the following paragraphs.

**Table 7-17: Landside Pressure Relief System Feature Design Requirements**

| Seepage Control Feature | Associated Potential Failure Modes | Design Elements | Advantages | Disadvantages |
|---|---|---|---|---|
| Blanket drains and toe drains | • Throughseepage | • Alignment<br>• Spacing<br>• Depth<br>• Capacity | • Cost<br>• Captures critical flows from levee toe | • Maintenance<br>• Allows seepage<br>• Requires drain outlet |
| Trench drains and relief wells | • Underseepage | • Alignment<br>• Spacing<br>• Depth<br>• Capacity | • Flexible configurations<br>• Small right of way | • Cost<br>• Maintenance flushing<br>• Requires drainage |

**Figure 7-38: Throughseepage Pressure Relief Systems**

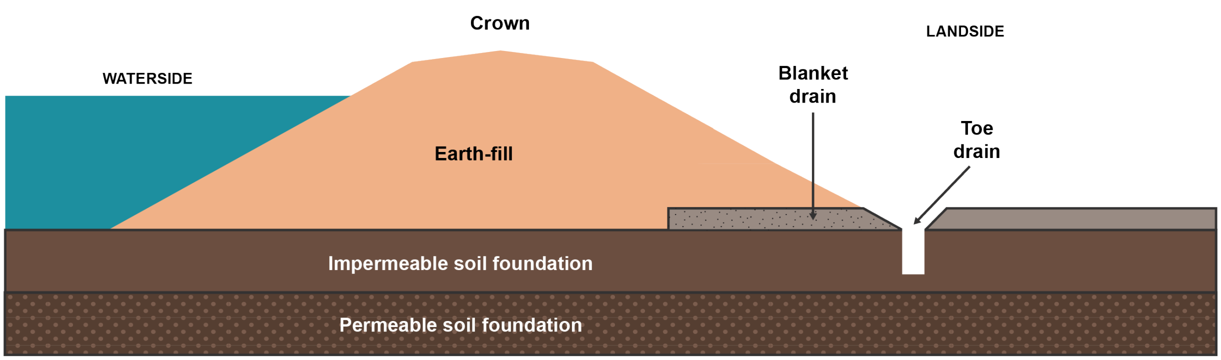
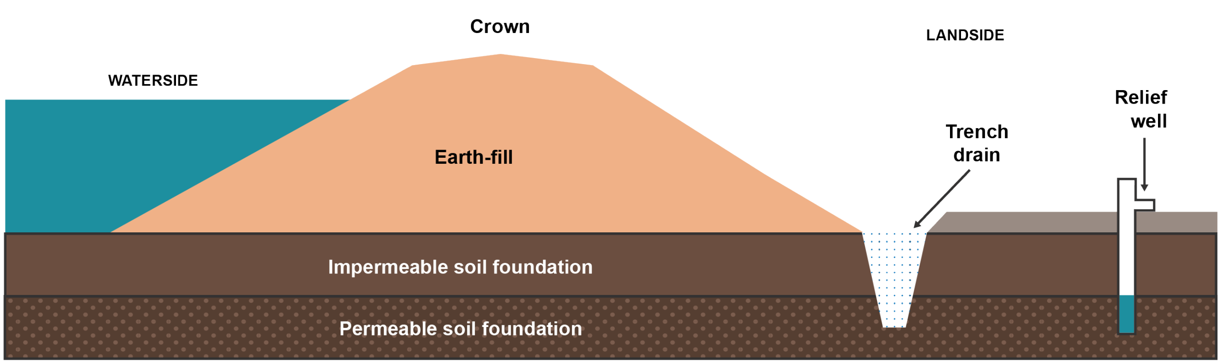



**Figure 7-39: Underseepage Pressure Relief Systems**




## 9.4.1 Alignment

The blanket drain and toe drain alignments should be along the landside toe, to capture seepage and relieve throughseepage pressures on the landside. The trench drain can be near the toe or located away from the toe as needed to relieve underseepage pressures on the landside. Relief wells are normally located away from the toe.

### 9.4.2 Spacing

All three types of drains should be continuous along the segment with seepage concerns. Relief well spacing should be determined by the estimated volume of seepage water to be intercepted to relieve pressure for the design water level (see **Chapter 6**). This should be estimated from the seepage analyses which will account for the head loss at the wells and determine water pressures midway between wells. Established design methods for determining relief well spacing are explained in EM 1110-2-1914 (USACE, 1992b).

### 9.4.3 Depth

The depth of the toe drains should be sufficient to collect throughseepage at the toe based on seepage modeling.

The depth of trench drains should be sufficient to penetrate into the permeable soil foundation through which underseepage occurs. However, the trench depth may be limited by excavation stability and/or the ability to drain by gravity from the trench and this may limit the use of trench drains in some situations

The depth of relief wells should be established based upon required capacity and the soil profile developed from a review of boring logs drilled at or near each well location. Capacity should be confirmed based upon pump tests performed after installation is completed.

### 9.4.4 Capacity and Materials

The required capacity of the systems should be estimated from the seepage analyses.

The material in drains should be filter-compatible with the in situ soil to avoid loss of soil into the drain via contact erosion or suffusion. Further details are available for review in EM 1110-2-1913 (USACE, 2000).

### 9.4.5 Advantages and Disadvantages

Landside pressure relief systems have the advantages of generally being lower cost than some of the alternatives. For modifications and rehabilitation, they are also often easier to install than modifying the levee itself.

The principal disadvantages of landside pressure relief systems are that they require routine maintenance and a drainage design to carry water away from the embankment. This can be challenging in relatively flat environments, where positive gravity drainage is difficult to establish.

Specific challenges for relief wells that should be considered are the following:

- Systems are prone to clogging and need to be cleaned and tested regularly.
- Potential for vandalism of relief wells.
- Need for well permits.
- Relief wells require pump tests every three to five years to verify their capacity. If the capacity reduces below a certain level, the well should be refurbished. If being refurbished does not restore well capacity, well replacement will be necessary.

While these are not insurmountable challenges, whole-life cost estimates should allow for well inspections, pump testing, and replacement of wells from time to time.

# 10 Controlled Overtopping, Channels, and Floodways

## 10.1 Locations of Controlled Overtopping

In some settings, the cost may be prohibitive to armor all reaches of a levee to resist overtopping as a resilience measure. As discussed in **Chapter 6**, rather than have overtopping occur at all locations simultaneously, or at locations that cannot be predicted in advance, it may be better for riverine levees to select locations in the formulation phase for initial controlled overtopping. By controlling the locations and preventing breach at those locations but allowing river water levels to be lowered, the magnitude of flood inundation within the leveed area may be reduced, resulting in lower life-safety and economic risk. In coastal settings, locations of controlled overtopping are not advised because the volume of water available for overtopping is unlimited and there is considerable uncertainty where storm surge and wave actions may affect a levee.

The key principles for design of a location of controlled overtopping are as follows:

- Capacity. Sufficient flood water should be released out of the river at such locations to fulfill the primary function of reducing river water levels upstream and/or downstream, elsewhere maintaining river levels below the levee crest.
- Resilience. The structure carrying the design overflow at the location of controlled overtopping for the anticipated duration should perform without significant deterioration or structural failure. Overflow at such locations will be infrequent and therefore the performance of the structure should be robust given the erosive power of overflowing water and that malfunction may lead to serious and unpredicted flooding elsewhere.
- Diversion of water. The likely destination for the water which overtops the levee should be one where the water can be contained and managed safely, normally an alternative channel or a safe area of temporary storage. The frequency of overtopping at such locations should be taken into account in the design and include assessment of the impact of controlled overtopping on the receiving area.

Various types of gates may be fitted at locations of controlled overtopping to risk reduction and regulate the flow discharged. The discharge characteristics of the gates and associated structures and their operation will determine the amount of water passing over the levee. Where gates are used to control rates of overtopping, they may include several gate bays separated by piers. These piers commonly support a bridge or walkway that facilitates the process of gate opening. The piers contain the necessary hardware required to retain the gates and any equipment needed to adjust gate settings. The gates may be engineered to permit overflow in extreme conditions.

Hydraulic design at locations of controlled overtopping consists of two interacting components:

1. Assessment of the impact of the removal of the overtopping water on the remaining flood hydrograph in the channel.
2. Calculation of the flow behavior at the location of the controlled overtopping itself.

The design process is an iterative one and may involve various kinds of computational models and even physical models for final optimization.

The levee surface at the location of controlled overtopping should be designed to carry the range of possible overflows without failure or significant deterioration, given the anticipated durations. Typically, the surface is concrete, riprap, grass, or some variation of these. This requires consideration of:

- Levee surface details.
- Structural integrity of both the levee and the surface protection system.
- Durability of the materials.
- All interfaces (e.g., drainage or bedding layers between the surfacing immediately beneath the overtopping flow and the body of the levee).

Structures at locations of controlled overtopping typically consist of three main parts:

1. A threshold that defines the crest level.
2. A slope that carries the water over the landward side of the levee.
3. A stilling basin that diffuses the energy of the overtopping water at or close to the toe of the levee.

Structures at locations of controlled overtopping need to be designed to carry varying volumes of water:

- During a minor flood, a relatively small volume of water needs to be discharged, and thus the crest structure could be short and only marginally lower than the rest of the levee crest.
- During a major flood, a much greater volume of water needs to be discharged, requiring either a longer crest (which would be expensive) or a lower crest (which would then spill water more frequently than may be ideal).

Lower local crest levels at locations of controlled overtopping often create transitions with the adjoining levee; the recommendations for design of transitions in section 8 should be followed.

## 10.2 Channels and Floodways

As described in **Chapter 2**, channels and floodways act as a diversion for riverine floodwater flows to be released into less critical areas. Such diversions may include

- Diversion of flood water from the river into the leveed area
- Diversion of water to from the leveed area to another area or basin which is either not prone to flooding and/or where other existing drainage facilities can be used to remove the water.

- Removal of water from a detention basin before the water in a basin rises to a level that can cause damage.

Where the alignment of the channel or floodway is such that it has to pass through the levee, provision should be made for an appropriate design of the crossing location. Regardless of the type of structure used to convey water across/through the levee, adequate channels should be constructed to convey the water to the outlets or control structures to avoid localized flooding. Furthermore, because a levee creates a barrier during flood events, some of the water on the landward side may need to be stored for later gravity discharge or pumping across the levee.

Controlled flow options for the design of the crossing at such locations involve the use of gates and weirs. Design considerations are similar considerations to those already described for closure structures (section 7).

Uncontrolled flow options include various types of fuses designed to be removed under flood conditions. These include:

- Weak fuses which overtop and are washed away (i.e., breach) under high flows. The design of such fuses must ensure that the rest of the permanent levee is able to withstand loss of the fuse material, including provision of any scour protection to the sides and foundation of the fuse.
- Fuses designed to be breached by explosives. A demolition plan should be prepared for such locations, with drill holes installed for the addition of explosive charges.

Additional guidance on best practices for design of channels and floodways is available as follows:

- For design analysis and criteria of design for channels that carry rapid and/or tranquil flows, see EM 1110-2-1601 (USACE, 1994b).
- For determining potential channel instability and sedimentation effects, see EM 1110-2-1418 (USACE, 1994a), or the most recent manual available. The manual aids in identifying the type and severity of channel stability and sedimentation problems, the need for and scope of further hydraulic studies to address those problems, and design features to promote channel stability.
- For the design of reinforced concrete-lined flood control channels which convey rapid and tranquil storm water flows, see EM 1110-2-2007 (USACE, 1995d).

# 11 Interior Drainage Systems

The levee reduces risk to an interior area from riverine or tidal flooding. Normally, provisions are made in the levee to pass runoff out of the leveed area, preferably by gravity through drainage pipelines and control gate structures if possible. However, during a flood or storm surge event, outlets are closed, and the backup of water to be drained in the leveed area can cause interior flooding from storm runoff. This can be exacerbated by water passing into the leveed area due to throughseepage, underseepage and due to overtopping of levees (including at locations of controlled overtopping, see the previous section). The flooding can be a risk to life, property, and infrastructure, and should be addressed as part of the levee design process.

Interior area drainage formulation studies are therefore an essential aspect of formulation and design of levees. As part of this activity and regardless of the type of structure used to convey water back across or through the levee, adequate ditches, channels (or pipes) should be constructed within the leveed area to avoid localized flooding. During flood events, some of the water on the landward side may need to be stored for later gravity discharge or pumping across the levee.

Although facilities and costs may be minor compared to the levee project, they potentially can affect many stakeholders. Furthermore, interior drainage studies can be complex, depending on the level of development in the leveed area. The extent of the analyses is scalable, based on the size of the interior leveed area and land use. For guidance on conducting interior drainage studies see EM 1110-2-1413 (USACE, 2018).

Penetrations through levees are potential weak points for a failure that could lead to a levee breach. Levee penetrations include various public/private utilities, drainage structures collectively termed pipes, and supporting ancillary structures.

Potential failure modes can develop because of improper choice of penetration materials, deficiencies in the design process and detailing of penetrations, deficiencies in construction, and other causes. Designers should identify and address all potential failure modes. Potential failure modes include:

- Longitudinal seepage along the exterior surface of the penetration or joint seal failures leading to progressive growth of voids in the levee (internal erosion).
- Structural failure of penetrations or ancillary features (e.g., valve vaults, gatewells).
- Settlement of penetrations and ancillary features.
- Shear failure and cracking at connection points to hard features (e.g., at gatewells and headwalls).

Further discussion and guidance on identifying and addressing potential failure modes associated with conduits, pipes, and culverts during the design is provided in EM 1110-2-2902 (USACE, 2020).

## 11.1 Pipes

Pipes include pump station discharge pipes, gravity drainage pipes, and ducts for utility penetrations to power appurtenances. Where these become penetrations (section 2.3.6) as they pass within or beneath an embankment, careful design is required to address potential failure modes. Detailed guidance on the design of pipes passing through levees is available in EM 1110-2-2902 (USACE, 2020).

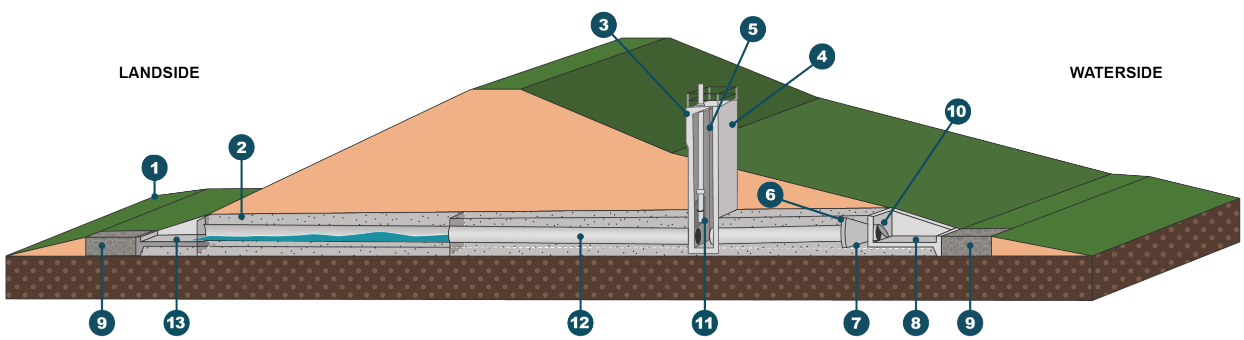
Special care should be given to pressurized pipelines to mitigate the chance of a blowout failure in the levee or foundation. Gravity outlets can be used to drain an interior area during a flood event, as long as the stage on the waterside is lower than the stage in the channel or collection pond on the landside. When the stage on the waterside exceeds the inlet water level, the gravity outlet on the waterside should be closed to prevent backflow. Any manually operated gates should ideally be located on the waterside edge of the levee crest in a vertical gate shaft and be accessible at all times.

Figure 7-40 shows a typical gravity drainage pipe penetration without a pump station. Figure 7-41 shows typical pipe penetrations with a pump station. There may be one or more pressurized pump discharge lines and a gravity drainage pipe used to bypass the pump station and drain water to the receiving body during non-flood periods.

Figure 7-40 shows a couple of features of importance:

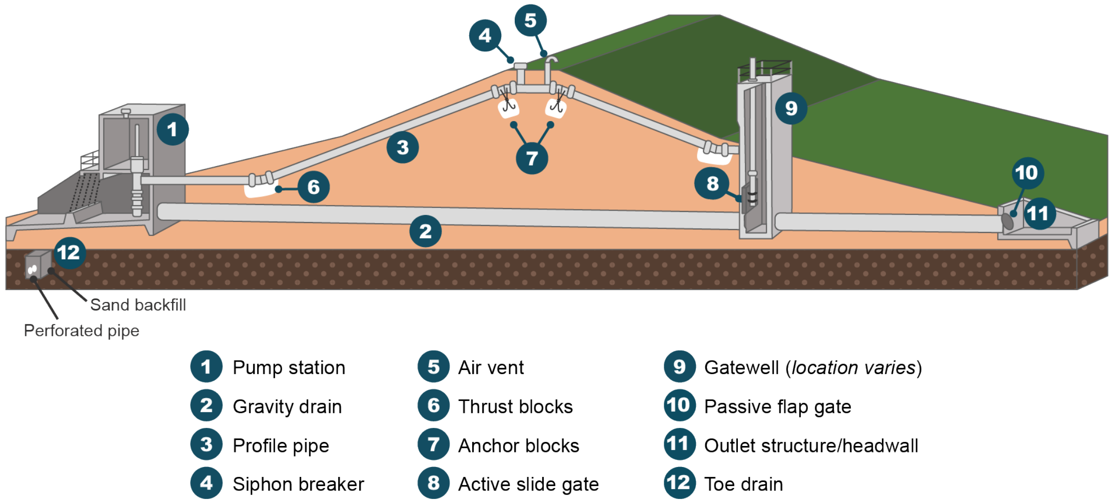
- Pipes should ideally not be bedded directly on compacted earth, but instead should be bedded on a controlled low strength material, which has self-consolidating and cementing characteristics.
- The outer surface of the pipe can act as a focal point for throughseepage during flood conditions leading to concentrated leak erosion. For this reason, a cone of filter material should be provided around the pipe at the landside end in two zones: filter diaphragm and filter transition. This drainage fill minimizes the transportation of fine soil material, provides controlled exit for water seeping along the pipe, and increases the integrity of the levee embankment.

### Figure 7-40: Typical Drainage Pipe Penetration—Embankment Levee



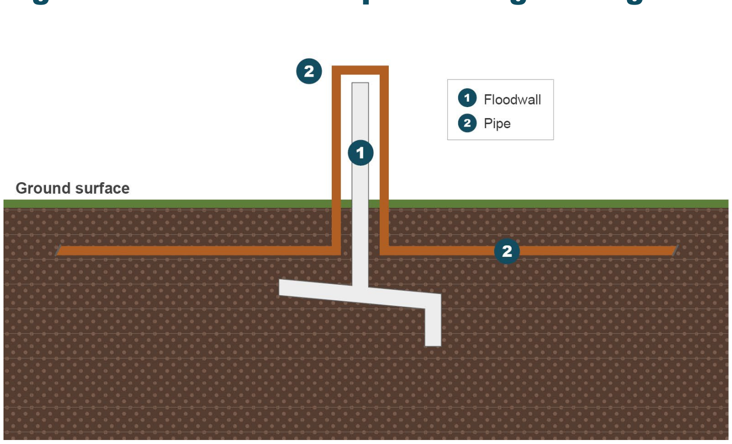

1 Existing ground surface
2 Controlled low strength material
3 Floor stands
4 Gatewell
5 Discharge pipes from pumping plants may terminate in gatewells
6 Filter diaphragm
7 Filter transition
8 Outlet structure
9 Riprap
10 Flap gate required for fast-rising streams
11 Sluice gate
12 Reinforced concrete pipe
13 Inlet structure for concrete pipe

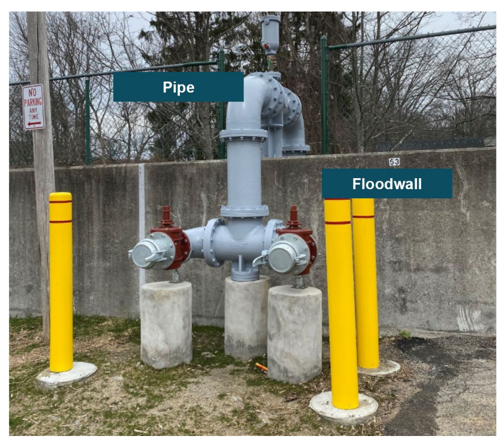
### Figure 7-41: Interior Pump Station Pipe Penetrations




Interior drainage pipe and utility penetrations may also be needed for floodwalls (concrete or sheetpile). Figure 7-42 shows typical installation details for the floodwall penetrations. For new interior drainage pipelines and existing utilities to remain, the critical design feature will be sealing the penetration against seepage during a flood while allowing for the possibility of movement (settlement or rotation) of the floodwall over time. The designer should consider avoiding penetration by routing pipes or utilities over the floodwall where practicable. For buried drainage penetrations, the design should include a discharge headwall with rock slope protection to prevent erosion. Extreme care should be used when bedding pipes on earth and consideration given to use of controlled low strength material for bedding.

### Figure 7-42: Detail of Pipe Passing Through Floodwall




*View of a pipe passing over a floodwall, rather than through the levee under the floodwall.*

### 11.1.1 Pipe Material

#### 11.1.1.1 Concrete Pipe

The following types of concrete pipe are typically used in levee applications:

- Round non-pressure reinforced concrete pipe manufactured according to American Society for Testing and Materials International C76; used with gasketed joints.
- Low-pressure reinforced concrete pipe manufactured according to American Society for Testing and Materials International C361 or American Water Works Association C302; used with gasketed joints. Both standards are interchangeable.

The internal pipe diameter for high-risk levees should be at least 48 inches to facilitate installation, maintenance, and inspection. Other levees may have a minimum diameter of 36 inches.

#### 11.1.1.2 Concrete Box Culverts

Reinforced concrete box culverts can also be used to convey drainage flows through levees. If used, however, box culverts should be cast-in-place with specialized design considerations for the joints. Precast box culverts should not be used because their joints have a history of leakage.

#### 11.1.1.3 Corrugated Steel Pipe Material

Corrugated steel pipe manufactured according to American Society for Testing and Materials International A760 and A796.2.2 with gasketed joints may be a potentially viable option within a levee; however, corrugated steel pipe should only be used for non-pressurized applications with properly designed bedding and backfill.

#### 11.1.1.4 Steel Pipe Material

Steel pipe conforming to American Water Works Association C200 with fittings conforming to American Water Works Association C208 may be used for gravity drainage pipes and pump discharge lines. Pipe is typically mortar lined and cement coated. Joints should be welded. Welded butt strap should only be used for field closures.

### 11.1.2 Pipe Design

The structural design of pipe penetrations should include all potential loadings, including earth loads (trench/embankment), road and railroad loadings, surface concentrated loadings, construction loads, and internal and external water pressure loadings. All potential loads should be identified and included in the design criteria, along with the proper methods used to analyze and design each feature and component. EM 1110-2-2902 (USACE, 2020), which covers conduits, pipes, and culverts for dams and levees, provides useful design guidelines, including material selection, design methods, loading combinations, safety factors, design details (e.g., trenching, bedding, backfill, use of controlled low strength material), procedures to control seepage along conduits, settlement and pipe connections to hard structures, and design examples.

Cast-in-place reinforced concrete box culverts, if selected, should be designed following the guidance in EM 1110-2-2104 (USACE, 2016b) with particular attention paid to the joint detail, since it is critical to ensure no soil intrusion will be possible (American Water Works Association, 2008).

Steel pipes should have corrosion protection provided either in the form of a resistant surface coating or by using a cathodic protection system. For concrete and steel pipe, specifications should require leak testing of joints as the pipe is assembled. Completed pipe should also be hydrostatic tested following American Society for Testing and Materials International C1103 (C13 Committee, 2022), or other applicable standard/guidelines.

Upon completion of installation, pipe interiors should be inspected and documented to prove a baseline condition for comparison with future inspections.

The specifications should require a detailed construction report be prepared with photographs and record drawings documenting all aspects of construction, including problems encountered, defects encountered, and corrective actions. This documentation is a valuable reference for future inspections, risk assessments, and evaluation of problems that might develop over the life of the project.

### INFORMATION SOURCES FOR PIPE MATERIALS AND DESIGN

The following documents provide useful information on pipe materials and design methods with design examples:

- American Water Works Association Manual M9, Concrete Pressure Pipe (American Water Works Association, 2008).
- American Water Works Association Manual M11, Steel Pipe, a Guide to Design and Installation (Dechant, et al., 2017).
- American Concrete Pipe Association, Design Manual, Concrete Pipe. (American Concrete Pipe Association, 1980).
- American Society of Testing and Materials International ATSM C76-22, Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe (C13 Committee, 2022b).
- American Society of Testing and Materials International ASTM C361-22, Standard Specification for Reinforced Concrete Low-Head Pressure Pipe (C13 Committee, 2022b).

## 11.2 Ancillary Components

Ancillary components associated with the penetrations may include headwalls, gatewells, and valve vaults. Mechanical components may include slide gates or sluice gates, passive flap gates, air vents, and siphon breakers. A drainage pump station could also be part of the project. This chapter includes guidance for ancillary components while section 12 treats pump stations.

Appurtenant components are important for the proper performance of drainage penetrations. Failure of certain components could lead to uncontrolled releases or levee breach.

For design considerations related to mechanical components, refer to Table 7-18 for references to best practices.

**Table 7-18: References for Design of Mechanical Components**

| Mechanical Component | Best Practice Reference |
|---|---|
| Sluice gate | • EM 1110-2-2902 Conduits, Pipes and Culverts Associated with Levees and Dams and Other Civil Works Structures (USACE, 2020b).<br>• Engineering Technical Letter (ETL) 1110-2-584 Design of Hydraulic Steel Structures (USACE, 2014).<br>• EM 1110-2-6054 Inspection, Evaluation, and Repair of Hydraulic Steel Structures (USACE, 2001b).<br>• EM 1110-2-3105 Mechanical and Electrical Design of Pump Stations (USACE, 1995). |
| Flap gate | • EM 1110-2-2902 Conduits, Pipes and Culverts Associated with Levees and Dams and Other Civil Works Structures (USACE, 2020).<br>• ETL 1110-2-584 Design of Hydraulic Steel Structures (USACE, 2014).<br>• EM 1110-2-6054 Inspection, Evaluation, and Repair of Hydraulic Steel Structures (USACE, 2001b).<br>• EM 1110-2-3105 Mechanical and Electrical Design of Pump Stations (USACE, 2020c). |
| Duckbill check value | • EM 1110-2-2902 Conduits, Pipes and Culverts Associated with Levees and Dams and Other Civil Works Structures (USACE, 2020). |
| Air vents and siphon breakers | • EM 1110-2-3105 Mechanical and Electrical Design of Pump Stations (USACE, 1995). |

### 11.2.1 Gatewells

Gatewells are typically reinforced concrete structures usually located on the waterside of a levee system. Applicable regulations may require the gatewell be located next to the waterside levee crest. Gatewells typically house active gates (e.g., slide gates or sluice gates) that are closed during flood events to prevent backflow through the pipe. Gatewells may be rectangular or circular. Precast concrete gatewells may be used in lieu of cast-in-place concrete, if designed and detailed to satisfy the loading and functional requirements of the levee system, and if the joints are designed to prevent soil infiltration. Figure 7-43 shows a typical gatewell.

The gate operator is located on top of the gatewell and connected to the gate via a shaft properly attached to the wall. If gates are motor operated, the gate shaft wall anchors should be designed to resist the stall torque specified by the motor manufacturer. Failure to do so may result in a failed closure system that could allow interior flooding.

### Figure 7-43: Example Gatewell with Sluice Gate and Operating Shaft

*View looking into a gatewell, with sluice gate at bottom.*

Waterside gatewells should have an operations platform (i.e., location of actuator or manual controls to lower the gate) a minimum of 1 foot higher than the height of the levee or floodwall to allow access at all times. Gatewells located immediately next to the levee crest provide the advantage of easy access regardless of the river level (**Chapter 9**); otherwise, a platform/bridge or an exterior ladder reachable by boat should be installed to access the top of the gatewell. There should also be access down to the bottom to the pipe and gate. If a bridge is constructed, consider that tall gatewells may require large piers within the embankment, creating the potential for additional seepage paths. For new construction, gatewells should be located at the waterside edge of the levee crest, eliminating the need for boat or platform/bridge access during a flood event.

All gatewell joints, whether cast-in-place construction joints or connections between stacked precast elements, should have a waterstop to prevent the infiltration of embankment material into the gatewell. The design of the waterstop should accommodate the anticipated differential settlement and resulting connection movement without failing. Similar to pump stations, movement of a gatewell due to instability or flotation could affect the performance of the levee by compromising the pipe gatewell. A poorly installed or compromised connection could allow material loss through the defect, leading to internal erosion. In cases where the pipe rests on a concrete cradle, the designer should determine if doweling the cradle and gatewell together is needed and/or appropriate. Typically, gatewells are very deep in order to service the connecting drainage pipe; therefore, the internal erosion process may be active for years, thus removing many yards of embankment material before a surface expression is observed for further guidance see EM 1110-2-2104 (USACE, 2016b) and EM 1110-2-2502 (USACE, 1989a).

### 11.2.2 Valve Vaults

The valve vault houses isolation valves for pump discharge. The vault is a reinforced concrete structure with grating or hatch covers, located next to the waterside edge of the levee crest. The valve may be a vertical slide gate or other commercially available full-flow valve. Gates or valves should be manually operable; motor operation can be added if desired. Air relief and vacuum relief valves, or a combination of air-vacuum valves, are also needed in combination with the gates or valves. A small access port with a bolted cover is also provided for video inspection of the pipe.

### 11.2.3 Headwalls and Gates

Headwalls should be installed wherever drain pipes enter or exit the levee toe, and at the landward end of pressurized pipelines. Gates should be provided at each headwall and may be of the various types discussed in **Chapter 2** (sluice or lift gate, flap gate, duckbill). Figure 7-44 shows typical headwalls with flap gates.

**Figure 7-44: Example Headwalls with Flap Gates**

*View of a headwall with a flap gate in Louisville, Kentucky.*

Headwalls function to recess the inflow or outflow end of a pipe into the fill slope to improve flow conditions, anchor the pipe, support gates, and control erosion and scour from the pipe outflow area. Headwalls are typically constructed using concrete and should use wingwalls for added stability. All new pipes should have a headwall on both ends. New landside headwalls should have sufficient area on their face to install gated drains associated with an internal seepage filter. To meet coverage requirements related to the internal filter, the height of the landside headwall above the pipe crown may have to be taller than most precast models; therefore, standard U.S. Department of Transportation headwalls may need to be modified.[2]

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[2] Reference Chapter 17 of EM 1110-3-136, EM 1110-2-2104 (USACE, 2016b), and EM 1110-2-2002 (USACE, 1995) for additional information and design requirements for headwalls.

# 12 Pump Stations

Pump stations are included in levees when gravity drainage pipe systems cannot be used to remove water from the leveed area.

If required, the nature and capacity of a pump station will depend on hydraulic calculations linked to the amount of water stored in the interior area under various water levels and the consequences of the associated inundation. Depending on the area of the interior drainage basin, multiple pump stations may be needed at strategically placed locations. The failure of a pump station during a flood could result in considerable damage within the leveed area, causing the loss of some or all of the benefits that justified construction of the project. Consequently, pump station dependability should be the primary consideration during the design and pump selection process.

Pump stations are beneficial to drain interior water when no means exist to add another type of outlet. As shown in Figure 7-41, a typical pump station includes a pump, a gravity drainage pipe, and discharge pressure pipelines that pass through the levee and a gated valve. The pump discharge pipelines should cross through the levee at an elevation above the levee waterside design flood level.

This section provides general guidance for designing the pump station itself. Comprehensive pump station design guidance for civil and structural design—including foundations, loads, safety factors, and design methods—is available in EM 1110-2-3102 (USACE, 1995e) and EM 1110-2-3104 (USACE, 1989b). The major mechanical and electrical equipment selected for use should be rugged, reliable, and well suited for the type of service. For guidance on the design of pump stations, see EM 1110-2-3105 (USACE, 2020c). The pump station structure (frequently reinforced concrete) should be sized to house and support the equipment with adequate room for O&M.

Best practices for design of other components of pump stations can be found in Table 7-19.

### Table 7-19: References to Design of Pump Station Components

| Pump Station Component | Best Practice Reference |
|---|---|
| Structural | • Unified Facilities Criteria (UFC) 3-310-04 Seismic Design for Buildings (USACE, NAVFAC, and AFCEC, 2013).<br>• EM 1110-2-3104 Structural and Architectural Design of Pumping Stations (USACE, 1989b).<br>• American Society of Civil Engineers 7, Minimum Design Loads for Buildings and Other Structures (ASCE, 2022).<br>• Federal Emergency Management Agency (FEMA) P-361, Safe Rooms for Tornadoes and Hurricanes: Guidance for Community and Residential Safe Rooms, Fourth Edition (FEMA, 2021).<br>• EM 1110-2-3400 Painting: New Construction and Maintenance (USACE, 1995f). |
| Mechanical | • UFC 3-410-01 Heating, Ventilating, and Air Conditioning Systems (USACE, Naval Facilities Engineering Command, and Air Force Civil Engineer Support Agency, 2013).<br>• UFC 3-600-01 Fire Protection Engineering for Facilities (USACE et al., 2016).<br>• Unified Facilities Guide Specifications (UFGS) 35 45 01 Vertical Pumps, Axial-Flow, and Mixed-Flow Impeller-Type (USACE et al., 2021).<br>• UFGS 35 45 02.00 10 Submersible Pump, Axial-Flow, and Mixed-Flow Type (USACE, NAVFAC, and AFCEC, 2021b).<br>• UFGS 35 45 03.00 10 Speed Reducers for Storm Water Pumps (USACE et al., 2022).<br>• EM 1110-2-3105 Mechanical and Electrical Design of Pump Stations (USACE, 2020c).<br>• EM 1110-2-1424 Lubricants and Hydraulic Fluids (USACE, 2016a).<br>• EM 1110-2-2704 Cathodic Protection Systems for Civil Works Structures (USACE, 1999).<br>• ETL 1110-2-327 Geometry Limitations for the Formed Suction Intake, (Fletcher, 1990). |
| Electrical | • UFC 3-520-01 Interior Electrical Systems (USACE et al., 2016).<br>• UFC 3-530-01 Interior and Exterior Lighting Systems and Controls (USACE et al., 2023).<br>• UFC 3-550-01 Exterior Electrical Power Distribution (USACE, NAVFAC, and AFCEC, 2016b).<br>• UFGS Section 26 29 01.00 10 Electric Motors 3-Phase Vertical Induction Type (USACE et al., 2022).<br>• UFGS Section 26 29 02.00 10 Electric Motors 3-Phase Vertical Synchronous Type (USACE, NAVFAC, and AFCEC, 2022b, p. 29).<br>• UFGS Section 26 41 00 Lightning Protection System (USACE, NAVFAC, and AFCEC, 2023b, p. 41). |
| Connection to interior drainage systems | • EM 1110-2-2902 Conduits, Pipes and Culverts Associated with Levees and Dams and Other Civil Works Structures (USACE, 2020). |

## 12.1 Design

### 12.1.1 Hydrologic Studies

An interior drainage study and hydrologic studies of the leveed area should be completed as part of the formulation process for the levee project (**Chapter 6**). These studies provide the basis for establishing pumping requirements at various water stages permissible in the leveed area. The studies also provide the basis for designing the pump bypass gravity drainage system if one is provided with the pumping station. Siting the pump station along the levee alignment should be decided in the formulation study as well. The preferred location for a pump station is at the landside levee toe, which reduces the height of the pump column and the depth of the building.

The design should verify that the drainage and hydrologic studies account for all of the flows that may require pumping. In addition to interior storm drainage, other flows may include discharges for seepage collection systems, from wave overflow and from limited levee overtopping, if allowed in the levee design.

### 12.1.2 Pump Station Type

Pump stations typically have a wet-pit sump and employ vertical mixed-flow or axial-flow pumps. Water is usually pumped directly from storage ponds, ditches, or channels. When practical, provision should be made for exclusion of water from the pump sump and for maintaining the sump in a dry condition during inoperative periods. The operating floor level supporting the pump motors should be set above the maximum water level expected on the inlet side. Pump discharges may be located below or above the operating deck level.

Depending on location, pump stations may be open air or have a building enclosure over the operating deck. Suitable access should be provided to the pump station for construction and for permanent access for O&M.

### 12.1.3 Sump Pit and Adjacent Levee Design

Sump pits in or adjacent to levees probably cut through the clay blanket and need to be specially considered in seepage design (section 9) and slope stability design (section 5.2). During a flood, the sump pit may well become a weak point along the levee with elevated hydraulic gradients. The best practice is therefore to restrict hydraulic gradients around a pump station sump during a flood to a maximum of 0.3, due to the inability to observe and effectively floodfight anything that does occur.

### 12.1.4 Pump and Motor Selection

The number and resulting size of stormwater pumps should be determined by an economic study. This study should consider the consequences and related costs due to flooding if one pump malfunctions during a flood event. The greater the number of pumps, the smaller the reduction of the total station capacity if one pump malfunctions. However, this increased protection results in higher equipment, facility, and O&M costs. The need to reduce the impact if one pump malfunctions will most likely be appropriate in urban areas where a pump failure could cause significant property damage and raise ponding more rapidly to life threatening depths. Further discussion and guidance on identifying and addressing potential failure modes during design is provided in EM 1110-2-2902 (USACE, 2020b).

### 12.1.5 Pump Controls

The decision as to the type of control to specify for a flood control pumping station should be based on providing maximum reliability consistent with economic design. In the majority of cases, controls providing for manual start and automatic stop will be the most economical. From the standpoint of reliability, such controls are preferred. However, some installations may find the use of automatic start and stop controls to be an advantage, such as where limited sump capacity and inflow conditions would make manual starting impracticable due to short operating cycles, or where economy is obtained by using pumps of assorted sizes operating in a predetermined sequence. The control circuits of automatic stations should provide protection against simultaneous starting of all pumping units following a power interruption.

### 12.1.6 Forebay and Sump Sizing

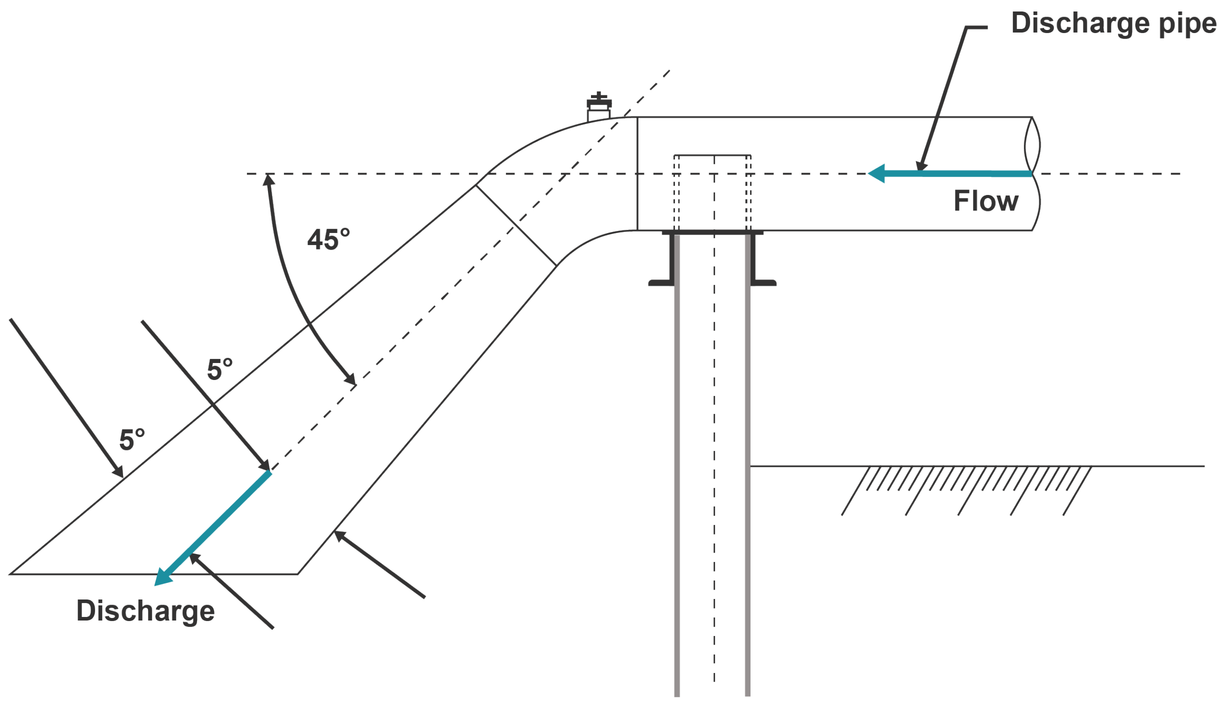
As a minimum, the size of the sump or pond serving the pumps may affect the selection of sizes and number of pumps. Pumps may cycle on and off as the water level in the sump varies up and down with the runoff. Provided sufficient storage so that the time between starts (cycle time) equals or exceeds the minimum operating cycle times to avoid damage to motors. Cycle times are based on motor power (kilowatt and horsepower) and are typically provided by motor manufacturers. EM 1110-2-3102 (USACE, 1995e) provides additional guidance on sizing sumps and forebays.

### 12.1.7 Pipes

All piping within a pump station structure should be ductile iron or steel with flanged joints or welded joints. In general, a single discharge pipeline should be installed for each pump. On large lines with submerged outlets, the discharge should be terminated in a cone to reduce flow velocity and corresponding exit losses (Figure 7-45). Downturn angle may vary but the flare of the discharge cone should be limited to 10 degrees maximum (five degrees off centerline) to avoid flow separation. Discharge piping should be constructed of steel or ductile iron, although high-density polyethylene piping may provide some advantages for conduit over levee sections where significant settlement is expected. The feasibility of high-density polyethylene should be evaluated during the design phase. All discharge line pipe should be protected on the inside with a smooth coating. Buried pipe should also be provided with an outer protective coal-tar coating and possibly wrapping. Shop coatings should be used to the maximum extent possible due to the enhanced quality control.[3]

[3] Refer to EM 1110-2-3105 (USACE, 2020) for best practices in pipe intake and discharge construction.

**Figure 7-45: Discharge Cone or Bell for a Pump Station**




## 12.1.8 Trash Racks

All flows into drainage pumping stations should be screened before reaching the pumps. Conventional bar screens (trash racks) are the preferred method of screening. Suction strainers should be avoided as they clog readily and are difficult to clean. Trash racks should be located to allow incoming flows to pass through the rack before reaching any pump intake, flow to be evenly distributed over the submerged rack surface and raking to be accomplished coincident with pump operation.

Trash racks should have ample net area so that the velocity of the flow through the gross rack area does not exceed 2.5 feet per second. The clear opening between bars should be approximately 1 3/4 inches, but should not exceed 3 inches. Bar spacing should be coordinated with the pump manufacturer.

## 12.1.9 Spare Equipment

Spare equipment should be considered for equipment whose design is unique or one-of-a-kind in construction, which would make replacement lengthy or very costly. Spare equipment for most pump stations should consist of bearings, impellers, shaft sleeves, temperature probes, relays, switches, lubricators, and any other types of auxiliary equipment being used on the pumping unit. Equipment problems caused by condensation and exposure to sewer gases in pumping stations used to pump sanitary sewage and storm water require additional corrosion resistant materials and sealants. Spare equipment should also include a spare impeller and pump bowl section if there are a large number of pumping units at the project (typically five or more) or there are multiple stations using the same size and type of pump procured under the same contract. Spare parts should also be provided for the prime mover.

## 12.2 Power Supply

All facilities necessary to supply the electric power required to operate the pumping stations should be provided as part of the flood risk reduction project. Power supply should be coordinated with the utility providing the service to determine the extent of work needed to supply the power and what components would be a cost to the project. The construction required may vary from the simple overhead service drop at the pumping station site at utilization voltage to extensive installations involving transmission lines, switching, and transformer equipment. The substation should be located and constructed so that access is available to the electric utility for maintenance and repair.

In general, flood protection pumping stations should be considered emergency facilities. Equipment and power supply should be selected primarily on the basis of reliability under emergency conditions. The need for additional emergency or standby power supply facilities should be considered.

## 12.3 Other Considerations

The design should include instrumentation to monitor and record water levels at the entrance to the station and in the receiving water body. This can be incorporated into the monitoring and control system for the station. Flow rate monitoring in the pump discharge piping should also be considered.

For enclosed pumping stations, lighting, heating, and ventilation will be needed. Removable roof hatches should be provided to remove and replace pumps and motors if needed.

The primary cause of equipment deterioration in many pumping stations is simply from lack of operation and long durations of downtime and associated moisture problems caused by this downtime. These conditions should be considered when preparing designs and specifications. The designer should investigate the use of heaters in the housings of motors, motor control centers, and switchgear to help mitigate moisture condensation.

- For example, bearings and seals on pumps can deteriorate if not used or exercised on a regular basis. Excessive moisture in operating buildings can lead to rust and corrosion in electrical cabinets. The designer should give preference to those materials that require the least maintenance and have the longest life.
- Specifications covering the materials and construction considered best suited to meet the usual service conditions should be provided for various pump station equipment.

Pumping stations are critical infrastructure. Security needs should be evaluated and implemented in design.

# 13 Instrumentation

## 13.1 Instrumentation and Monitoring Plan

An instrumentation and monitoring plan for the project should be prepared (Figure 7-46), taking into account the recommendations in **Chapter 9**. The extent of instrumentation included in the plan should be informed by the results of the risk assessment (**Chapter 4**). In higher risk situations it may be justifiable to complement other information and analysis by including sufficient instrumentation that will help reduce uncertainty in the understanding of levee behavior both during construction and during flood loading. In lower risk situations, extensive instrumentation is less likely to be justified. However, in either situation, if an observational approach is adopted in order to limit construction costs, instrumentation will be necessary in order to validate the eventual design.

The instrumentation and monitoring plan should be prepared in advance of the commencement of construction so that the construction management team understands how to install and monitor instrumentation, what action limits will apply, and what actions will be necessary to meet the designer's intent. Monitoring during construction ensures adverse conditions do not develop that can jeopardize the work or endanger workers or the public. The plan should indicate the required instrumentation, the timing for placement and baseline readings, threshold action levels, and who is responsible for installation and reading. Common instrumentation for levee projects may include piezometers, inclinometers, and settlement gages or plates. Installation locations and details should be clearly identified on the plans.

This plan generally does not include environmental monitoring, such as noise or air quality. These items should be provided by the constructor as required by the contract documents.

**Figure 7-46: Information to Include in a Monitoring Plan**

**INFORMATION THAT SHOULD BE DEFINED IN THE PLAN WOULD INCLUDE:**

- **Specifying the duration and parameters to be monitored.**
- **Identifying the data collection and management techniques.**
- **Setting guidelines for relocating instrumentation in the field, if needed.**
- **Identifying qualifications for the staff installing instruments and taking readings.**
- **Monitoring frequency and threshold action limits, actions required.**
- **Identifying the budget requirements.**
- **Selecting the appropriate instrument.**
- **Identifying reporting requirements.**

## 13.2 Instrumentation of Embankments

Monitoring instrumentation installed during levee embankment construction and post-construction may include:

- Seepage or groundwater monitoring wells/piezometers for water level or pressure measurements.
- Settlement plates to monitor construction movement of the cutoff wall systems.

- Earth pressure cells to monitor cutoff wall (soil contact stress).
- Inclinometers or tiltmeters to monitor displacement of the embankments.
- Flood water elevation gages.

Piezometers and settlement monitoring measures can be designed in accordance with EM 1110-2-1908 (USACE, 2020a). Access to read monitoring instruments during high water events should be considered in design.

Construction site monitoring may include requirements for visual observation or inspection or automated monitoring via camera or video. Project sites commonly are monitored using artificial intelligence software and ground, mounted, or drone-based cameras.

## 13.3 Instrumentation of Floodwalls and Structures

The instrumentation for the floodwalls and associated structures should be monitored during construction and post-construction. Instrumentation and monitoring for walls that are part of embankment dams or levees are described in EM 1110-2-1908 (USACE, 2020a). Measurements of movements and pressures furnish valuable information for use in verifying design assumptions. Most importantly, the data may forewarn of a potentially dangerous situation that can affect the post-construction stability of the floodwall or structure.

Settlement reference markers installed to monitor movements should be tied into a permanent baseline, located so it is unaffected by movements of the wall. When establishment of a baseline is not feasible, the relative movements observed between floodwalls and adjacent structures can provide valuable data on behavior of the wall. For floodwalls 15 feet or shorter, the settlement reference markers alone should be adequate. For taller floodwalls, use of inclinometers or tiltmeters should be considered.

## 13.4 Post Construction Monitoring

The designer may require instrumentation monitoring be continued beyond the end of construction. In this case, the plan should indicate who will assume post-construction monitoring responsibility and how collected data from construction should be transferred.

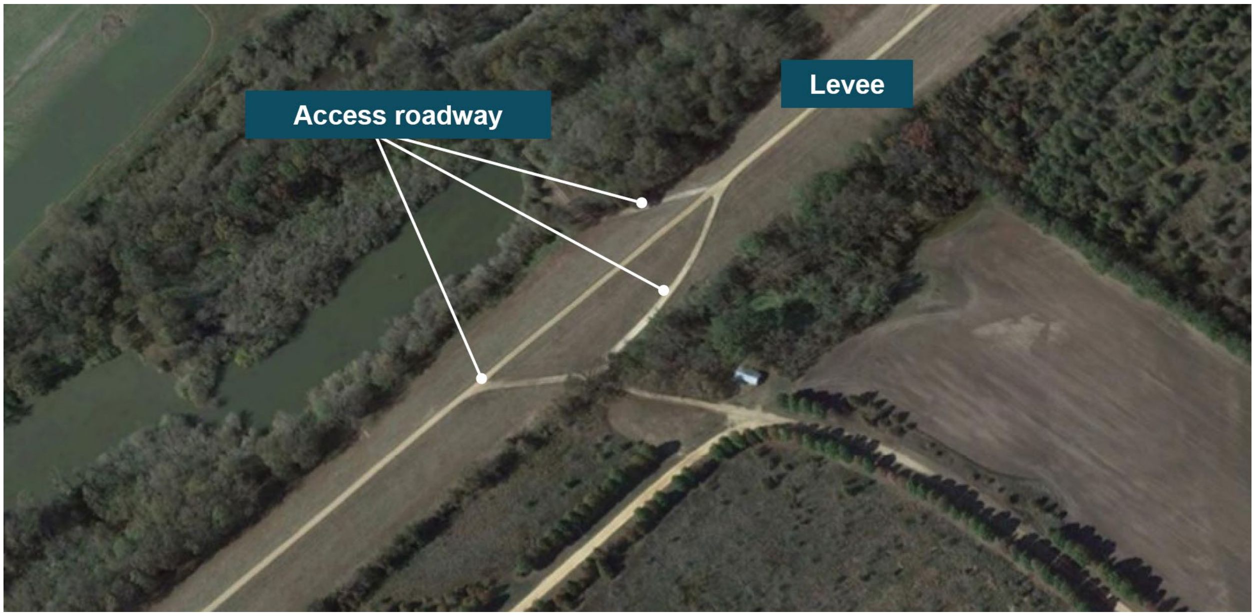
## 13.5 Water Level and Tide Gages

The design should include locating automatic recording water level gages for creeks and rivers adjacent to the levee system and tide gages for coastal levee systems. These provide needed real-time information to inform operation of the levee. This information will be particularly important in making decisions regarding operating flood protection closure devices.

# 14 Levee Access

Where possible, access should be provided to the levee at reasonably close intervals from public roads or using private roads with a negotiated easement in place for inspection, maintenance, and floodfighting operations. If possible, these roads should be all-weather roads. Figure 7-47 shows typical access roadways to the levee crest.

### Figure 7-47: Example Access Roadway




*Access roadway to levee crown of Mississippi River levee north of Vicksburg, Mississippi.*

Access roads should be provided on the levee crown for operations, maintenance, and floodfighting operations. Access roads also should be provided for levee structures and appurtenances, including gate or closure structures and pump stations, and be adjacent to relief wells. This type of road should be surfaced with suitable gravel or a crushed-stone base course that permits vehicle access during wet weather without causing detrimental effects to the levee or presenting safety hazards to levee inspection and maintenance personnel. Non-woven geotextiles or geogrids may be used under aggregate surfacing to improve subgrade stability, which may reduce maintenance and improve the ability for vehicles to navigate the road during inspections and flood-fighting operations.

Turnouts should be used to allow for a means for the passing of two motor vehicles on a one-lane access road on the levee. Turnouts should be provided at intervals of approximately one-half mile. They are particularly beneficial where no ramps are within the reach. Turnarounds sometimes are provided to allow heavy equipment to reverse direction on levees.

Ramps should be installed approximately every mile to permit vehicular traffic to access onto and exit the levee crown, and to connect the levee crown with the landside and waterside toes of the levee. Ramps on the waterside of the levee should be oriented to minimize turbulence. Ramps should be angled for side-approach instead of at a right-angle (perpendicular to the levee access road) in order to reduce the requirement for additional embankment material. The ramp width should be determined based on its intended function. The grade of the ramp should be no steeper than 10%. Side slopes on the ramp generally should be the same slope ratio as the adjacent embankment slope and should not be steeper than 1 vertical and 3 horizontal, to allow grass-cutting equipment to operate. The ramp should be surfaced with suitable gravel or crushed stone. The levee section should never be reduced to accommodate a ramp.

# 15 Summary

As critical infrastructure reducing flood risk, each levee should be designed, modified, or remediated following the current best engineering practices applicable at the time of design. This also includes:

- Appropriately characterizing site conditions, including the reach-by-reach variations.
- Ensuring the design delivers the required level of flood risk reduction, including designing the levee features and the transitions between them as a complete system.
- Delivering an economically feasible approach by optimizing the balance between costs, risks, and benefits.
- Delivering a design which is constructable, while communicating the design intent clearly through the appropriate construction documentation.

The level of investigation and study adopted in the design process should be risk-informed (i.e., scaled to the level of flood risk) and should be scaled to the size and nature of the works. For example:

- For a simple repair, the preliminary design may consist of a few sketches (of a couple of options) and a few notes, put together by an experienced individual following a site visit.
- For a large project involving a new levee through the center of a town, the design process would usually be more extensive, requiring the consideration of a range of options, flood risk assessments, environmental impact assessments and detailed drawings and specifications supported by potentially complex calculations.

Levees should be designed for whole-life resilience, which includes:

- Preparing for both present day and for future hazard loading conditions on the levee systems (such as those associated with climate change).
- Ability of the levee and its components to absorb adverse loading without failing through any of the potential failure modes, including failure during overload conditions exceeding the nominal design conditions. This includes consideration of providing resilience in parts of the levee system to allow the levee to overtop without breaching.
- Ability to readily restore or recover the levee after damage due to adverse loadings.
- Building in the ability to strengthen/adapt the levee to meet future changes in hazards or within the leveed area. (This may include adjusting the land-take to allow for future change.)

Ecological/environmental risks and opportunities should be considered through the design process, incorporating the natural environment into the design wherever possible.

Social risks and opportunities should be considered through the design process, where possible implementing design features that support the everyday functioning of the local community.

Related content associated with this chapter is included in detail in other chapters of the National Levee Safety Guidelines as described in Table 7-20.

### Table 7-20: Related Content

| Chapter | Chapter Title | Related Content |
| --- | --- | --- |
| 1 | Managing Flood Risk | • Levee form and function<br>• Types of levee projects |
| 2 | Understanding Levee Fundamentals | • Levee features |
| 3 | Engaging Communities | • Engaging for levee projects |
| 4 | Estimating Levee Risk | • Flood and levee risk<br>• Risk assessment |
| 5 | Managing Levee Risk | |
| 6 | Formulating a Levee Project | • Levee alignment<br>• Crown elevation and geometry |
| 7 | Designing a Levee | |
| 8 | Constructing a Levee | • Instrumentation |
| 9 | Operating and Maintaining a Levee | • Instrumentation and monitoring |
| 10 | Managing Levee Emergencies | |
| 11 | Reconnecting the Floodplain | |
| 12 | Enhancing Community Resilience | • Community resilience |